ROCKY MOUNTAIN ARSENAL

DENVER, COLORADO

SUPPLEMENT TO

REPORT ON

PROGRAM FOR RECLAMATION OF SURFACE AQUIFER

PREPARED BY
C. E. JACOB
GROUNDWATER CONSULTANTS
NORTHRIDGE, CALIFORNIA

FOR

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA
JANUARY 1961

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THIS REPORT IS A FINAL FOLLOW-UP TO A PRELIMINARY STUDY DONE BY THE CORPS OF ENGINEERS: PRELIMINARY STUDY GROUND WATER CONTAMINATION, RMA, 11 MAY 1960. THE BASIC REQUIREMENTS FOR THIS STUDY AS OUTLINED BY THE CHEMICAL CORPS INCLUDED THE FOLLOWING: 1. PLAN FOR CORRECTION OF CONTAMINATION, 2. DETERMINATION OF NATURE AND EXTENT OF CONTAMINATION, 3. DESCRIPTION AND NATURE OF PHYTOTOXICANTS, 4. DETERMINATION OF ANIMAL TOXICITY, AND 5. SUPPLEMENTARY METHOD OF WASTE DISPOSAL. DURING THE PREPARATION OF THIS REPORT, CHEMICAL COPRS. IMPLIED THAT THE TOTAL RESPONSIBILITY FOR THE COMBINED EFFORTS OF ALL AGENCIES INVOLVED IN THIS WORK HAS BEEN ASSIGNED TO THE CORPS OF ENGINEERS, THOUGH NO SPECIFIC INSTRUCTIONS TO THIS EFFECT HAVE EVER BEEN RECEIVED. THOUGH THERE ARE PROBLEMS TO BE SOLVED, ALL EFFORTS OF ALL AGENCIES ARE AIMED AT THE FINAL SOLUTION OF THE PARAMOUNT PROBLEM: DECONTAMINATION OF THE AQUIFER.

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GROUNDWATER, TREATMENT, DISPOSAL, WELL CHARACTERISTICS, PUMPING TEST METHODS, DRILLING, SALT BALANCE

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FIGURES Contd.

FIGURES Contd.

INTRODUCTION

Terms of Reference

This Supplement to the report on Program for Reclamation of Surface Aquifer, Rocky Mountain Arsenal, has been prepared by the firm of C. E. Jacob, Groundwater Consultants, Northridge, California, acting as Architect-Engineer under Contract No. DA-25-066-eng-6361.

Under this contract the Architect-Engineer is authorized to perform the following services:

(Article I-B - Statement of Services)

"1. Advise this office on collection of water-level and water-quality data in the field and also on the evaluation and study of this data.

"2. Initiate mathematical analyses, model studies or numerical calculations of the following problems:

a. Dispersion of salt water and other contaminants reaching the water table from infiltration or evaporation ponds.

b. Hydraulics of thin unconfined groundwater systems on curved and sloping bottoms.

c. Influence of anisotropy, inhomogeneties and stratification upon flow of groundwater in such aquifers and upon dispersion of contaminants therein.

d. Hydraulics of line of reclamation wells arranged down-gradient, or cross-gradient.

e. Hydraulics of other arrays of reclamation wells.

f. Hydraulics of other feasible reclamation schemes such as trenches.

g. Dispersion of distributed contaminant upon attempted withdrawal through various reclamation-well or trench systems, also residual of contamination under each system and distribution thereof.

"3. Test Drilling. Advise on most efficient spacing of test borings to define bedrock and outline extent and properties of aquifer. The positioning of these borings will be done where possible to permit their use in pumping tests of existing or specially con-

constructed pumping wells.

"4. Geophysical exploration. Advise on interpretation of geophysical surveys conducted to map bedrock between borings, whether by electrical resistivity or by seismic methods.

"5. Pumping-test methods. Outline procedure and techniques for determining character of aquifer by well pumping tests. Insofar as possible, tests will be run first using the existing production wells, such as abandoned farm wells still equipped with pumps in nearby areas. Design of pumping-test layouts requiring specially constructed production and observation wells will be planned by Architect-Engineer for the Omaha District.

"6. Leakage of ponds. Advise on methods of testing Pond F and other ponds, if desired, for water-tightness. This may be done by restricting inflow and observing water-level decline, or by using tracers if possible.

"7. Water balance. Undertake to reconcile best estimates of historic and present inflow to groundwater system and displacement of contaminated water.

"8. Salt balance. On the basis of latest geologic data and groundwater data plus distribution of salts revealed by program of sampling, make an inventory of various signigicant ions (such as chloride). These salt balances will be reconciled, if possible, to the aforementioned water balance. Attempt will be made to explain the past and present contamination in the light of the known characteristics of the aquifer and of its geologic structure as revealed by test drilling and sampling.

"9. Restricting the flow at arsenal boundary. Undertake studies of various methods proposed to restrict flow of contaminants underground at Rocky Mountain Arsenal. These analytical studies may be supplemented by model studies or by electronic analog computations.

"10. Reclamation of aquifer. Different schemes for removing contaminated groundwater through wells or open excavations will be evaluated. The possibility of injecting fresher water after pumping the contaminated water to accomplish dilution of the water in the ground will be studied. This will require analytical and electronic analog studies of the further dispersion of the contaminants upon attempted withdrawal."

Conferences

Conferences between the Architect-Engineer and personnel of the Omaha District and of the Missouri River Division of the Corps of Engineers were held in Omaha on the following dates: 6 July, 9 August, 20-21 October, and 6 December, 1960. During these conferences the Architect-Engineer advised the Corps of Engineers on Items 1, 3, 4, 5 and 6 under Article I-B of the contract.

The initiation of mathematical analyses and numerical calculations, under Item 2, is covered in Appendices E through I in this Supplement.

The outlining of the procedures and techniques of pumping tests, under Item 5, is covered in Appendices B and C of this Supplement.

The water balance and salt balance, under Items 7 and 8, were discussed in the Preliminary Report and are reviewed in this Supplement.

Methods of restricting flow of contaminants at the arsenal boundary and of reclaiming the aquifer, under Items 9 and 10, are covered in the five alternative schemes outlined in the report itself. These schemes were considered in detail during the conference of 6 December 1960 in Omaha.

Preliminary and Progress Reports

A Preliminary Report on Ground Water Problem, Rocky Mountain Arsenal, was written 1 August 1960. That report contained one table and one plate and 13 pages of text, reproduced in blue-line prints. Progress reports in the form of Memoranda of Conferences were prepared 9 August 1960 and 21 October 1960, and were reporduced in blue-line prints.

AQUIFER CHARACTERISTICS

The hydraulic characteristics of the aquifer were determined by a number of well tests run in Area A (the contaminated area extending northward from the northwest boundary of the Arsenal). Table S-1 gives a summary of these formation constants. In the first column the kind of test is indicated and in the second the number or name of the wells tested. In column 3 values of hydraulic conductivity (K) in feet per day are given and in column 4 values of the storativity (S). Pertinent remarks are given in column 5.

For the locations of the wells tested see Figures D-1a and D-1b in Appendix D.

Ten wells shown on Figure D-1a were given drawdown-recovery tests. The results were doubtful in two cases, the Marty Well and the Matsumoto Well 2. The hydraulic conductivity determined by the remaining eight tests ranged from a low of 1,040 to a high of 2,640 ft/day. The average of the eight tests was 1,483 ft/day. (We shall round this off at 1,500 ft/day for the purpose of calculation.)

The data on the second line of Table S-1 refer to the interference test of the Aden Well. For further details see Tables D-4 and D-5 in Appendix D.

According to the logs of the six piezometers or observation wells (see Figure D-47b) the groundwater is generally unconfined in the vicinity of the Aden Well. The logs of three of the observation wells show "gravelly sand" or "sandy gravel" at the level of the water table, whereas two of the other three show "clayey silt" or "clayey sand". In the sixth observation well (P-6) about five or six feet of sandy clay is logged at and just above the water table. Despite these variations shown in the logs, a condition of unconfinement is reflected in the variable and high values of apparent storativity obtained from this interference test.

From the average drawdown and recoveries observed in the six observation wells the hydraulic conductivity is calculated to be about 1,230 ft/day. The initial value of apparent storativity is 0.056 or 5.6 percent. By extrapolating the trend of apparent storativity it is estimated that the maximum value would be on the order of 0.35 or 35 percent.

The data on the third line of Table S-1 relate to the interference test on the Marty Well. According to the logs plotted on Figure D-48b, at the time

TABLE S-1

SUMMARY OF FORMATION CONSTANTS FROM WELL TESTS

[1] Kind of test	[2] Well(s)	[3] Hydraulic conductivity K/[ft/day]	[4] Storativity S	[5] Remarks
Drawdown-recovery	eight	1,500	--	Ten tests in all; two doubtful
Interference	Aden	1,230	0.056	Avg. initial value
			.35	Max. ultimate value
Interference	Marty	1,330	.085	S fairly const. in time
Interference	Powers	1,090	.10	S fairly const. in time

of the test the water table stood in "sand" or "gravelly sand" in all six observation wells.

The average hydraulic conductivity from the drawdown and subsequent recovery observed in all six observation wells is 1,330 ft/day. (See Table D-6.) The storativity remained fairly constant throughout the period of the test and averaged about .085 or 8.5 percent. Undoubtedly the storativity would increase upon prolonged pumping or drainage.

The data on the fourth line of Table S-1 relate to the interference test of the Powers Well. Logs of four observation wells drilled near the Powers Well are shown graphically on Figure D-23b. In three of those four wells "clay" or "silty clay" was encountered at or just above the water table and in the fourth one a layer of "silty sand". At the time of the test the water table stood in "silty sand" or "gravelly sand" beneath these semi-impervious layers.

The average hydraulic conductivity was 1,090 ft/day and the apparent storativity about 0.10 or 10 percent. The storativity was fairly constant in time over the period of drawdown and also over the subsequent period of recovery.

In summary, for determining scale ratios for digital or analog computation we shall use 1,500 ft/day for the hydraulic conductivity and 0.20 or 20 percent for the average storativity.

WELL CHARACTERISTICS

Table D-1 summarized the drawdown-recovery tests run in ten wells near Rocky Mountain Arsenal. The average depth of the ten wells is 37.2 feet. The most common casing diameter is 48 inches and the most common material concrete. The eight wells whose tests were successful had an average discharge of about 215 gpm or 41,400 ft^3/day. The average hydraulic conductivity (see column [12] of Table D-1) is 1,483 ft/day or approximately 1,500 ft/day.

As a result of the step-drawdown test on the Powers Well it was concluded that "for practical purposes, over the range of discharge involved here, when dealing with wells similar in design to the Powers Well, the well loss may be neglected." (See Page D-11.)

For the purpose of determining scale ratios for analog or digital computation, then, we may use the effective well radius determined from the drawdown-recovery tests, its unnaturally small value making allowance for the transitional flow near the well.

Referring to column [15] of Table D-1, it is seen that the mean value of the logarithm of the quantity ($S^* r_w^2/h^*$) in feet is 0.72-5. Taking for the value of h^* an average initial depth of flow of 10.3 feet (the average for the eight successful tests), the logarithmic mean value of the product ($S^* r_w^2$) is about 5.5×10^{-4} ft^2. This is the figure that we shall use in determining scale ratios for electronic analog or for other models of the wells.

CHARACTERISTICS OF CUT-OFF TRENCH

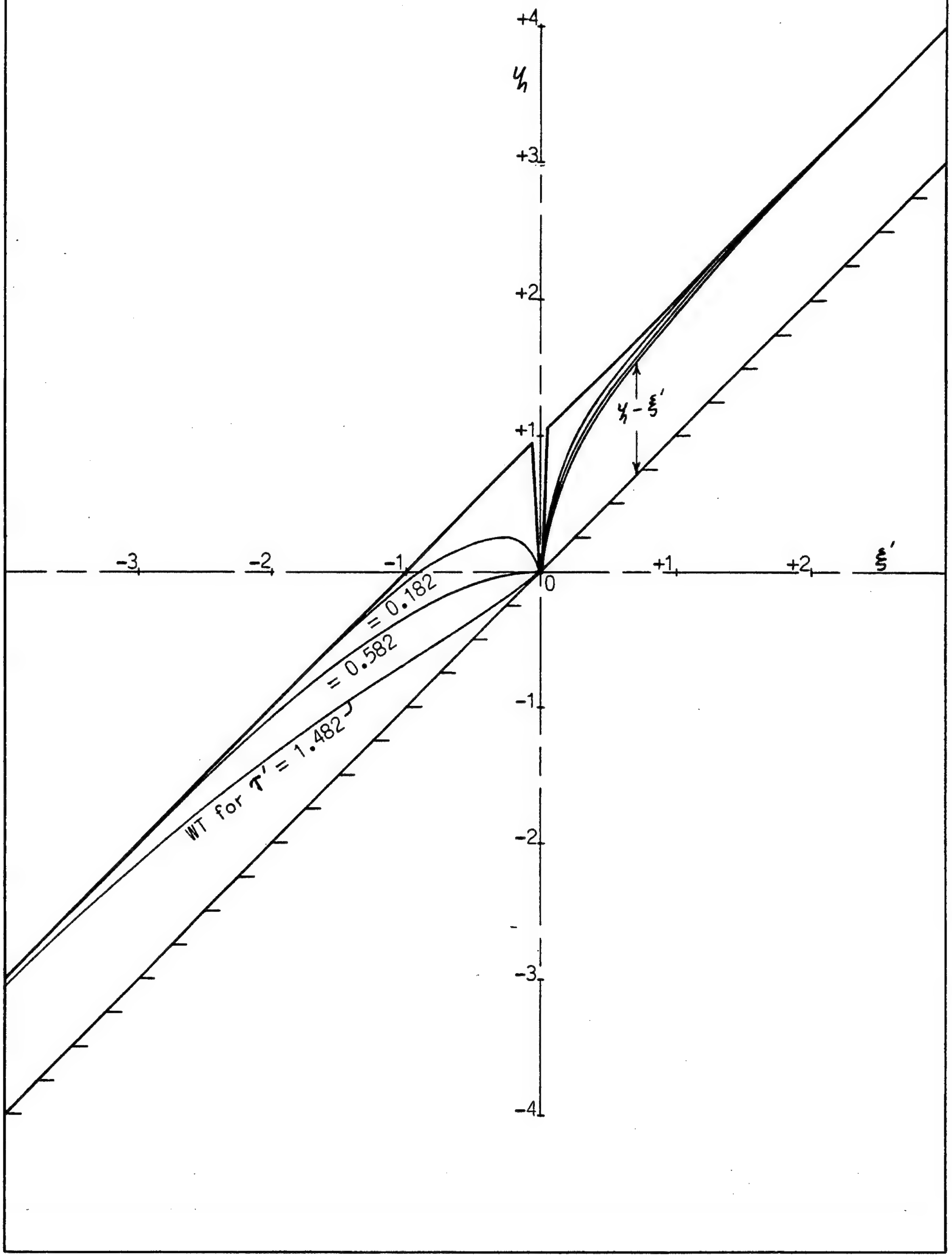
Figure S-1 shows a series of water-table profiles upstream and downstream from a cut-off trench excavated to the bottom of an inclined aquifer. These profiles were obtained by numerical integration of Equation G-3 in Appendix G. To interpret them in physical terms it is necessary to have an estimate of the scale factors involved.

If the initial depth of flow is about 10 ft on the average and if the slope of the water table is about 1/260 (See Plate 7), then the physical distance (horizontal) is related to the corresponding nondimensional space coordinate as follows: $x = \xi' \; 10 \text{ ft}/(1/260) = \xi' \; 2{,}600 \text{ ft}$. Similarly the head measured above the inclined bottom of the aquifer is related to the corresponding nondimensional coordinate as follows: $(h - z) = 10 \text{ ft} \; (\eta - \xi')$. (See Appendix E and Glossary for meaning of symbols.

The physical time is related to the corresponding nondimensional time coordinate as follows: $t = \tau'(Sh_o/\alpha^2 K) = (0.20 \times 10 \text{ ft} \times 67{,}600/1{,}500 \text{ ft/day})$. Here we have assumed that on the average S = 20 percent and K = 1,500 ft/day. It turns out, then, that t = (90 days) τ'.

Referring again to Figure S-1, it is seen that after about 0.6 units of time on the τ'-scale the gradient downstream from the trench is reduced to zero. Thereafter the flow from downstream back into the trench is zero, and provided the head or water level in the trench is kept at or below the bottom of the formation, there will be zero flow from the trench into the aquifer downstream.

WATER TABLE PROFILES NEAR CUT-OFF TRENCH
IN INCLINED AQUIFER



WATER BALANCE

A check may be made of the rate of movement of the contaminated ground-water and of the time that might have been required for it to traverse the distance from Reservoir A to the South Platte River. Water-table contours on Plate 2 and the profile on Plate 7 show an average water-table slope or hydraulic gradient of about 20 ft/mi. The average hydraulic conductivity from the drawdown - recovery tests is about 1,500 ft/day. Thus, assuming the average porosity is 35 percent, the average velocity of movement is as follows

$$v = (20 \times 1{,}500 \text{ ft/day})/(0.35 \times 5{,}280) = 16 \text{ ft/day}$$

At this rate, without dispersion it would take the water about 320 days to traverse a distance of one mile. To traverse the 6.4 miles distance from Reservoir A to the River it would take about 2,100 days or 5.7 years. Actually the most highly contaminated water has not yet reached the river. (See Plate 4.)

The foregoing calculation applies to water of average chloride concentration (about 1,100 ppm). By longitudinal dispersion of the advancing front contaminated water of lower concentration reaches the river much sooner. However, by transverse dispersion of the advancing front the advance is delayed.

Although evidently the contaminated water has been migrating over a period of about 17 years, a more detailed water balance than is now possible would probably show that much of the contaminated water has gone into storage by lateral dispersion on either side of the advancing thread or stream flowing from Reservoir A to the River. The influence of farm irrigation wells in helping to spread the contamination laterally must also be considered.

SALT BALANCE

Estimates of volumes and concentrations of contaminated water in the aquifer under and near Rocky Mountain Arsenal given in Table S-1 may be summarized as follows:

On the Arsenal	436 million cu ft averaging	1, 680 ppm chloride
Off the arsenal		
in Area A	301	720
in Area B	245	560
Total	982 million cu ft averaging	1, 100 ppm chloride

We may estimate the quantities of inflow of given concentrations needed to produce the foregoing estimated volumes of contaminated water. These symbols will be used:

V = volume of water

γ = specific weight of water, weight/volume

c = concentration of ion

t = elapsed time of flow

Q = rate of flow, volume per unit time

W = weight of ion in water volume V

The relationship between the weight of any given ion in a given volume of water is given by $W = c\gamma V$. Dividing both sides of the equation through by time we get the following relationship $W/t = c\gamma V/t = c\gamma Q$. Solving for the rate of flow of water, or volume per unit time, we get the following $Q = W/c\gamma t$.

To make a rough check on the chloride balance it will be assumed that water has been entering the water table and contaminating the groundwater over a period of 17 years, or about 6,000 days. The specific weight of water will be taken to be 62.4 lb/ft^3. From the foregoing table the volume of contaminated water is seen to be about 7.34 billion gallons or about 1 billion cubic feet. Thus, $W = c\gamma V = 1{,}100 \times 10^{-6} \times 62.4\ lb/ft^3 \times .980 \times 10^9 ft^3 = 6.7 \times 10^7$ lb. With an average chloride concentration of 1,100 ppm, the total weight of chloride ion is estimated to be 67,000,000 pounds.

Assuming that the average chloride content of the contaminating fluid were 4,000 parts per million, the average inflow rate to the water table needed

to produce the estimated volume of contaminated water would be as follows. (In this calculation the initial chloride content of the groundwater of the area, thought to be on the order of 50 ppm, is neglected.)

$$Q = W/c\gamma t = (6.7 \times 10^7 \text{ lb})/(4{,}000 \times 10^{-6} \times 62.4 \text{ lb/ft}^3 \times 6{,}000 \text{ day})$$
$$= 4.5 \times 10^4 \text{ ft}^3/\text{day} = 235 \text{ gpm} \pm$$

Thus it is seen that the average input to the water table of water having an average chloride content of 4,000 ppm would have had to be about 235 gpm. If the chloride content had been only 2,000 ppm on the average, the average rate of inflow would have had to be about 470 gpm.

Petri [1956, p. 48] reports as follows: "Although liquid wastes have been discharged into the disposal reservoirs in Sections 26, 35, and 36 since 1942, no records are available of the amounts of liquid discharged. Measurements made in June 1955 by the Ralph M. Parsons Co. **** indicate that at that time the average input at the disposal reservoirs was about 570 gpm."

Obviously the volume rate of input of contaminated water to the water table has continuously been less than the volume rate of input to the disposal reservoirs. However, as relatively little chloride is stored in the abandoned disposal reservoirs or retained in the subsoil of their bottoms, the rate of input of chloride to the water table has probably equalled on the average the rate of input of chloride to the disposal reservoirs. Offhand it would appear that the concentrations and quantities of inflow in the past have probably fallen within the limits set by the foregoing calculation.

APPENDIX A

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APPENDIX B

THEORY AND PROCEDURES OF WELL TESTING

Definitions

In this report the standards of the Soil Science Society of America and of the American Petroleum Institute are accepted, and the following definitions of technical terms are adhered to.

Permeability. The property of a porous water-bearing material that relates to its ability to conduct water under a hydraulic gradient is known as its permeability. The author uses the definition of this term accepted in the pertoleum industry. Thus, the permeability is the horizontal volume-flow per unit area per unit time per unit horizontal pressure gradient of a fluid of a unit viscosity and unit specific weight. The flow may be expressed in cubic centimeters per square centimeter per second, the horizontal pressure gradient in atmospheres per centimeter, the viscosity in centipoises, and the specific weight in grams per cubic centimeter. With this choice of units, the unit of permeability is called the "darcy" (American Petroleum Institute, Recommended Practice for Determining Permeability of Porous Media, RP 27, Sept. 1952).

Hydraulic conductivity. In soil mechanics and foundation engineering the term permeability is often applied mistakenly to another constant. This second constant or characteristic typifies not only the porous material but also the fluid flowing through it and includes in addition to the permeability as defined above, the fluid factors viscosity and specific weight. In this report this product is termed "hydraulic conductivity" (Soil Science Society of America, Subcommittee on Permeability and Infiltration, August 31, 1951). In terms of the hydraulic conductivity Darcy's law of flow may be expressed as follows: The volume-flow per unit area is equal to the product of the hydraulic conductivity by the hydraulic gradient, or loss of head per unit distance in the direction of flow. The hydraulic conductivity is the permeability multiplied by the specific weight of fluid and divided by its viscosity.

Transmissivity. A convenient term used to describe the ability of a uniformly thick bed or aquifer to transmit water is its transmissivity. This is the same as the transmissibility of Theis,but since the water in the aquifer is transmissible while the aquifer itself is transmissive, we prefer the word "transmissivity" to decribe the aquifer. The transmissivity is the hydraulic conductivity miltiplied by the thickness of transmissive formation.

Storativity. The ability of an aquifer to store water is expressed by a number called in this report its storativity. This is the same as "coefficient of

storage" or "storage coefficient" of Theis. It is defined as the volume of water removed from storage under a unit surface area by a unit decline of head, or expressed otherwise, it is the volume of water stored under a unit surface area by a unit rise of head. In confined aquifers the storativity ranges from about 10^{-5} to 10^{-3}, in unconfined aquifers from about .05 to .40. Intermediate values imply semi-confinement.

Pumping level is the water level observed inside a well when it is discharging.

Standing level is the water level observed inside a well when its pump is idle.

Static level is the water level that would have been observed at any time in a well if it had never been pumped.

Specific capacity is the ratio of the discharge to the drawdown it produces, measured inside the well and expressed in gpm/ft or in cfs/ft.

Specific drawdown is the ratio of drawdown to the discharge that produces it. It is the reciprocal of specific capacity and may be expressed in ft/gpm or in ft/cfs.

Residual drawdown. The difference at any time between the standing water level observed in a well after the same well or a neighboring well has been shut drown and the extrapolation of the standing water level from the next preceding period of recovery.

Theory of Well Tests

The theory of well tests is outlined in several papers that are listed in the Bibliography. See especially Jacob [1950], Jacob [1947], and Cooper and Jacob [1946]. The object of these tests is three-fold--- First, to determine the formation constants; second, to determine the characteristics of the pumping well; and third, to enable predictions of the future performance of the well.

Darcy's law. The law of horizontal flow of homogeneous fluids through porous media, such as water through sandstone, known as Darcy's law, is as follows: $Q = (kA/\mu)dp/dx$, where Q is the volume-rate of flow of water through area A under a pressure gradient dp/dx. The symbol μ stands for the viscosity of the water, and the symbol k stands for the permeability of the material through which the water is flowing. By dimensional analysis it may be seen that the unit of k is length squared (l^2). The standard unit of permeability is the darcy.

The foregoing equation applies only to horizontal flow. When the flow is in some general direction the driving force producing the flow is expressed in terms of head (h). Also, it is found convenient to lump together the permeability of the material through which the water flows, the specific weight of water and

also its viscosity, to give the following relationship: $Q = KA\, dh/dx$, with $K = \gamma k/\mu$ and $h = z + p/\gamma$.

Transmission of water. Many water-bearing formations in nature are of almost uniform thickness, in which case it is found convenient to lump together the factor K, hydraulic conductivity, with the thickness of the formation to form the coefficient known as "transmissivity", as already stated. Thus $Q = Kabdh/dx = aTdh/dx$, where $T = Kb$. In this last equation, a is the width of a strip of the water-bearing formation or aquifer at right angles to the direction of the flow (x); b is the average thickness of the aquifer.

Storage of water. The foregoing coefficients relate to the ability of an aquifer to transmit water. Another important characteristic of an aquifer is that which relates to its ability to store water. This is called the "storage coefficient" (Theis) or "storativity", already defined above.

Aquifers may be divided into two kinds-- confined aquifers and unconfined aquifers. Water may be stored and removed from storage within an unconfined aquifer by the rising and falling of the water table. The volume of water that goes into storage and comes out of storage is a fraction of the total space occupied by the water-bearing material. In coarse-grained materials this specific volume may approach the porosity of the material. In fine-grained materials, owing to capillary action the volume of water going into and coming out of storage may be somewhat less than the actual porosity. The ratio between the volume of stored water and the volume of space it occupies, known as the storativity is defined by the following equation: $S = dV/Adh$, where dV is the volume of stored water under a surface area A, and where dh is the differential of head accompanying the storage of a differential volume dV.

In confined aquifers water is stored through the compressibility of the water and the expansibility of the solid framework of the aquifer. It comes out of storage through the expansion of the water and the concomitant compression or compaction of the solid framework.

Well Testing Procedures

Interference tests. When a well that has been idle suddenly begins pumping at a constant rate from an aquifer of uniform thickness and uniform permeability, the drawdown at some distance r away from that well and at some time t since the start of pumping is given approximately by the following equation: $s = (2.30\, Q/4\pi T) \log (2.25\, Tt/Sr^2)$, where S and T are the storativity and transmissivity of the aquifer, respectively, and Q is the discharge rate of the well. It is seen from this equation that, by plotting s against t at any constant r,

the points should fall on a straight line on a semi-log paper, the linear scale of which is used for plotting drawdown (s) and the logarithmic scale of which is used for plotting time (t). Moreover, from the slope of the straight line drawn through such points plotted on such a semi-log graph, it should be possible to determine the transmissivity (T) using this relation: $T = 2.30\ Q\Delta \log t/4\pi\Delta s$, where $\Delta s/\Delta \log t$ is the slope of the straight line on the semi-log graph. For convenience, $\Delta \log t$ may be taken to be equal to 1, in which case Δs is the change of drawdown over a ten-fold variation of t, or over one "log cycle".

By extending the straight line back in time to where it intercepts the line of zero drawdown (or time axis), it is possible to determine S as follows: $S = 2.25\ Tt_o/r^2$, where t_o is the time at the intercept on the zero drawdown line.

Drawdown tests. The drawdown inside a pumping well is made up of two components -- the first, called "formation loss", is the loss of head from that great distance where the drawdown is negligible up to the face of the well, and the second, the "well-loss" is the loss of head accompanying the flow of water through the perforations in the casing and upward inside the casing to the pump intake. The formation loss is given by an equation similar to that given previously except that the effective radius of the well (r_w) is substituted in place of r. Then writing the well-loss as CQ^2 the drawdown in a pumping well may be expressed as follows: $s_w = (2.30\ Q/4\pi T)\log(2.25\ Tt/Sr_w2) + CQ^2$. This equation may be abbreviated as follows: $s_w = A(t)Q+CQ^2$, where A(t) is the formation-loss coefficient, which varies with time, and where C is the well-loss coefficient, which is constant. [See Jacob, 1947.]

It may be seen from the last two equations that a semi-log plotting of drawdown against time should again permit the determination of T, though not of S. Observation wells at known distances are needed to get S.

Step-drawdown tests. By pumping a well at different rates over successive periods of time and observing the trend of drawdown during each period it is possible to determine the well-loss coefficient, C, and the formation-loss coefficient, A, which is a function of the duration of each step. This kind of test is known as a "step-drawdown test".

Dividing the foregoing equation for drawdown by the discharge of the well, $s_w/Q = A(t) + CQ$, the ratio s_w/Q being the specific drawdown of the well. A plotting of specific drawdown against discharge for several steps of a test should give a straight line, the slope of which gives the well-loss coefficient, C, and the intercept of which gives the value of formation-loss coefficient for the duration of step that was used.

The formation-loss coefficient is also a function of the transmissivity, being inversely proportional to T. Thus it may be written $A = A'/T$. Then, dividing the drawdown equation by Q^2 instead of by Q, one may write $s_w/Q^2 = A'/TQ + C$. In this case, when plotting s_w/Q^2 against $1/QT$, the slope gives A' and the intercept gives C.

Not only may the foregoing plottings be used to analyze the data from step-drawdown tests of single wells, but they may be used to analyze statistically the data from simple drawdown tests (one-step drawdown tests) of many wells. Thus the average characteristics of a large number of wells of similar design may be determined.

Recovery tests. When a well that has been pumping continuously for some time is suddenly shut down, the water surface inside the casing begins to recover, and continues to recover for some time at a continuously diminishing rate. It is just as though the well were to continue discharging and a recharge well of the same strength were to be superimposed on top of it at the time of shutdown. In other words, the position of the recovering water surface is the composite of two effects, the drawdown and the superimposed recovery. This kind of test is known as a Theis recovery test (after C. V. Theis of the U. S. Geological Survey, see paper by Theis, Trans. A.G.U., vol. 16, p. 520, 1935). The residual drawdown at time t' after shutdown and time t after the start of pumping is given by this equation: $s' = (2.30\ Q/4\pi T) \log (t/t')$. The logarithms of the square of the distance and of the constant factors would appear twice in the equation for residual drawdown, but oppositely signed, and therefore they actually disappear from the equation.

Tests in thin unconfined aquifers. The foregoing methods apply to confined aquifers and also to unconfined aquifers, provided the maximum drawdowns are a small fraction of the original or undisturbed depth of flow and provided the storativity is fairly constant. When the drawdown is a large fraction of the undisturbed depth of flow the hydraulic conductivity is determined by plotting values of h^2 against (log r). Knowing K, values of S may be calculated. As S is less likely to vary with r than with t, calculated values of S averaged over the range of r are extrapolated in time to estimate the ultimate value of S. See Jacob [1950, p. 384].

APPENDIX C

FURTHER DETAILS ON WELL TESTING PROCEDURES

There follows a brief summary of procedure in the well tests. This is given to supplement the material found in Appendix B. That appendix outlines the theory of groundwater hydraulics applicable to well tests and briefly discusses the procedures of such tests. However, it was felt desirable to elaborate on these procedures somewhat. The references given in Appendix A in the Bibliography under the heading "Well Tests" will prove useful guides in this connection, especially the text by Muskat and the chapter in the book "Engineering Hydraulics" edited by Hunter Rouse. Chapter V of that compendium, entitled "Flow of Groundwater," was written by the author.

The drawdown-recovery tests should be run on as many wells as possible in the contaminated area that runs northward from the northwest boundary of the Arsenal, especially in Sections 9, 10, 15, 16, 21, and 22 of T2S, R67W. Those wells in that area still equipped with pumps and prime movers may be tested with little expense. Where it is possible to provide a valve to throttle the well, step-drawdown tests may be run in addition to the usual drawdown-recovery test. After these tests have been run and analyzed it will be possible to select certain ones of them for further testing by the interference method. This will require the drilling of a few observation wells at each site. Thereafter it may be found desirable to run interference tests at specially planned locations, having drilled especially for that purpose new production wells which might later be converted into reclamation wells to decontaminate the aquifer. However, this kind of test should be deferred until all other possibilities have been exhausted.

Drawdown-Recovery Test

This kind of test requires no observation wells but does require accurate measurements of discharge and drawdown in the pumping well itself, both before and during the test. See pages B-4 and B-5 of Appendix B.

Measurements of discharge. The discharge should be measured by means of a circular orifice on an extension of the discharge pipe. Where the discharge pipe is threaded at its end this may easily be extended. In other cases it may be necessary to provide a Dresser coupling followed by a valve for the purpose of throttling the well for step-drawdown tests, and then a length of pipe equal to about 10 diameters, on the end of which the orifice is placed. If it is not possible to install an orifice, the discharge perhaps could be measured by a sharp-edged weir in a weir box especially constructed for the purpose so as to be movable from site to site. Or the discharge might possible be measured volumetrically in a tank that could be transported from site to site. However, the orifice method is to be preferred.

Measurements of drawdown inside the pumping well should be made either by an electrical probe or by a steel tape. In most instances it should be found possible to enter the well through the bushing in the pump base provided for that purpose. A steel tape that is somewhat rusty will give a better water line than a new tape. The tape may be covered with carpenter's chalk to make the water line visible to the nearest hundreth of a foot.

Procedure. After a period of rest, during which the standing water level is measured repeatedly to establish its trend, the discharge of the well is begun and the trend of the drawdown during the period of pumping is observed by frequent measurement. The discharge also should be checked frequently to see whether it declines, especially during the early part of the pumping period.

After pumping at a fairly uniform rate for an hour or two the pump should be shut down abruptly, and the recovery of the standing water level inside the well should be measured frequently.

Graphical treatment. A semilogarithmic plotting of the drawdown and of the recovery against the logarithm of the time elapsed since the change in pumpage that produced that drawdown or recovery should enable an estimate of the transmissivity and hydraulic conductivity of the aquifer. Owing to the fact that the storativity of the aquifer will vary in time and will vary in a different manner during drawdown than during recovery, the interpretation of the graph may not be easy.

Step-Drawdown Test

Valved discharge. For a step-drawdown test accurate means of measuring discharge and drawdown inside the pumping well is required, as also a valve for the purpose of controlling closely the discharge.

Measurements of drawdown and discharge should be carried out the same way as in the drawdown-recovery tests.

Procedure. After the period of rest during which the standing water level in the well is measured repeatedly to establish its trend, the discharge of the well is begun at a fraction of its full capacity, say one third, and allowed to continue for about one hour. During this first period of pumping the trend of the drawdown is observed by frequent measurement. At the end of the first period of pumping the valve is abruptly opened to raise the discharge to say two thirds of its full capacity. Again the new trend of the drawdown is observed and extrapolated over the third period, at the beginning of which the throttling valve would be opened wide to allow the pump to operated at full capacity.

Graphical treatment. The data from the step-drawdown test should be plotted on a graph like Figure 1 in the preliminary report. It will be found that the formation loss is curvilinear. By the theory, already briefly outlined, it should be possible to divide the total drawdown at different rates of pumping into formation loss and well loss. However, because of the variable storativity of the aquifer some special interpretation may be required.

Interference Test

For each interference test at least four observation wells should be drilled penetrating sufficiently below the water table to allow for the drawdown during the period of pumping. These may be located two each on two lines, one line running up-gradient from the discharging well (or down-gradient if necessary) and the other running cross-gradient from the pumping well. On each line there should be a well about 50 feet from the pumping well and another about 100 feet away. If it is desired to get better results, then six observation wells should be drilled just penetrating the water table, three on each line, one 50 feet away, one 100 feet away, and one about 200 feet away. In those places where the aquifer is thicker, it may be desired to get still sharper results by having pairs of observation wells at each observation-well site, one penetrating just sufficiently far below the water table to allow for the drawdown during the pumping period and the other reaching to the bottom of the aquifer. However, in most instances this will not be necessary.

Measurement of drawdown and discharge. The same care must be taken in interference tests to observe accurately the discharge of the well. Pumping by electrically driven turbine is preferred because of the constancy of the speed and therefore the near constancy of the discharge. However, the discharge should be observed sufficiently close to detect any declining trend.

The pumping would have to continue 24 or 48 hours in order to produce strong influences at the distance out to the farthest observation well.

Graphical treatment. The data from this test is best treated by plotting it on semilogarithmic paper, arranging the horizontal scale or logarithmic scale to record time elapsed since the start of pumping and the vertical scale or ordinate to record the drawdown in the different wells. Theoretically, if the storativity were constant and the formation of uniform thickness and homogeneous, the drawdown curves should be straight lines on such a graph. However, it is anticipated that especially because of the variableness of the storativity, the drawdown data will plot on curves. Special care must be taken in their interpretation. Special attention should be paid to the last paragraph on page B-5 of Appendix B, which deals with modifications of the theory required for tests in thin unconfined aquifers, in which the drawdown is a sizable fraction of the undisturbed or initial depth of flow.

A set of forms that have been found useful for tabulating the data from the pumping tests should be used. Space is provided on each for the calculations leading to determinations of transmissivity and storativity.

ANALYSIS OF WELL TESTS

Introduction

During September and October of 1960 several wells in Township 2 South, Range 67 West were tested. The locations of these wells are given on Figure D-1. Drawdown recovery tests were run on 10 wells. On one of these 10 wells a step-drawdown test was run, from which we determine not only the hydraulic conductivity but also the well-loss coefficient. Three of the ten wells were used as discharge wells in interference tests. Piezometers or "observation wells" were drilled at distances of about 50, 100 and 200 feet on two radial lines from each pumping well. From these three tests the hydraulic conductivity and effective average storativity were determined, and also the effective radius of the pumping well. (For explanation of symbols used here see page D-17.)

Drawdown-Recovery Tests

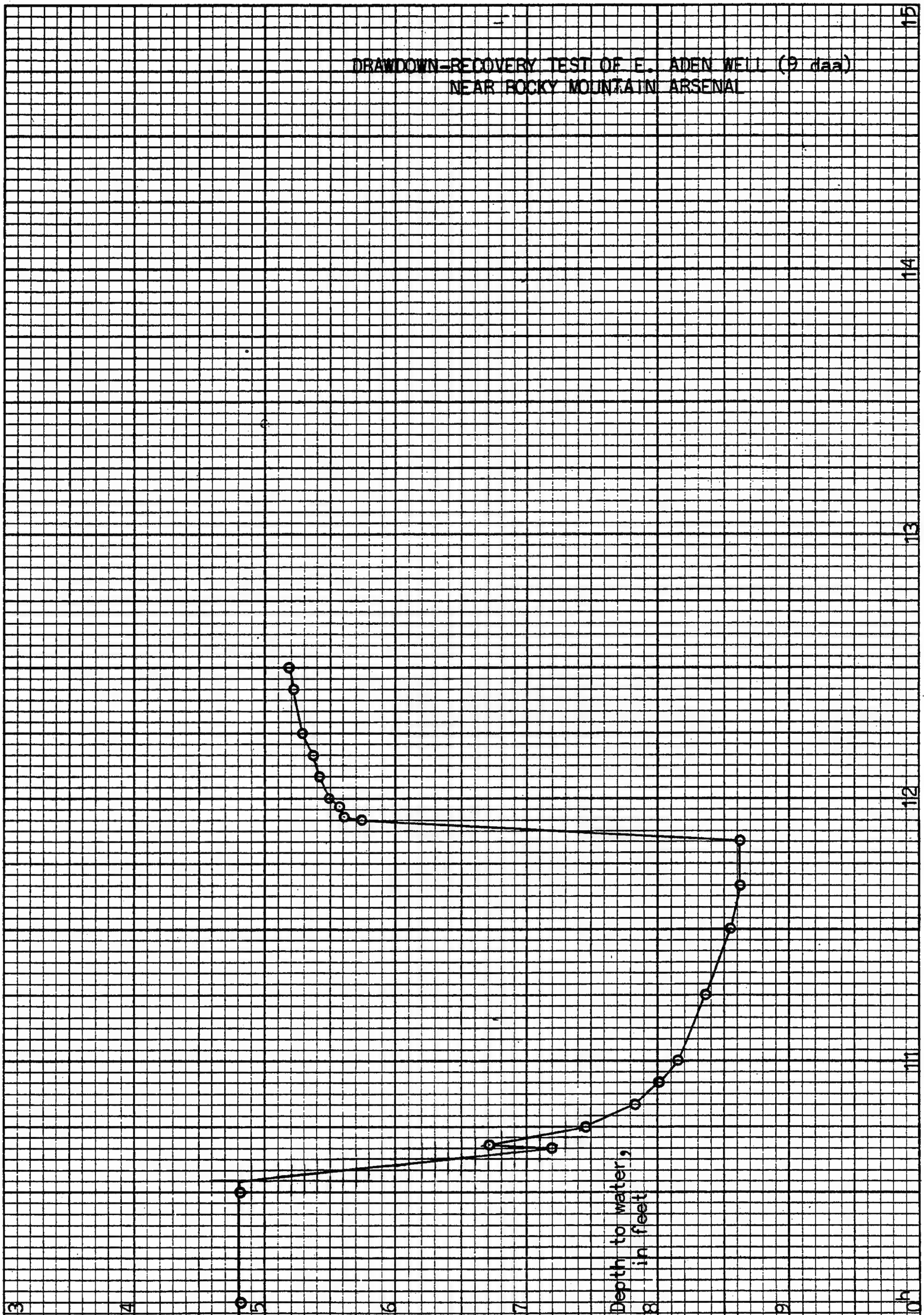
Theory and procedure. The theory of drawdown and recovery tests such as were run on the ten wells near Rocky Mountain Arsenal, are given in Appendix B, pages B-4 and B-5 and in the Memorandum of the Conference of 9 August, 1960 page 4.

Test of Aden Well. Figure D-2 is a water-level graph for the drawdown-recovery test of the Aden Well. Plotted thereon are readings of depth to water, in feet, versus clock time. Pertinent data on this test are as follows:

Date of test 17 September 1960.
Started pumping 10:37.
Stopped pumping 11:50.
Measured recovery until 13:25.
Discharge declined from 200 to 190 and averaged 194 gpm.
Discharge measured by 3-inch orifice.
Drilled depth not known.
Sounded depth 24.1 ft.
Measuring point 1 ft above ground.
Type of screen or perforated section not known .
Lenth of same not known.
For other pertinent data see Table D-1, first line.
Note: "No water came up on first start; we had neglected to prime pump."

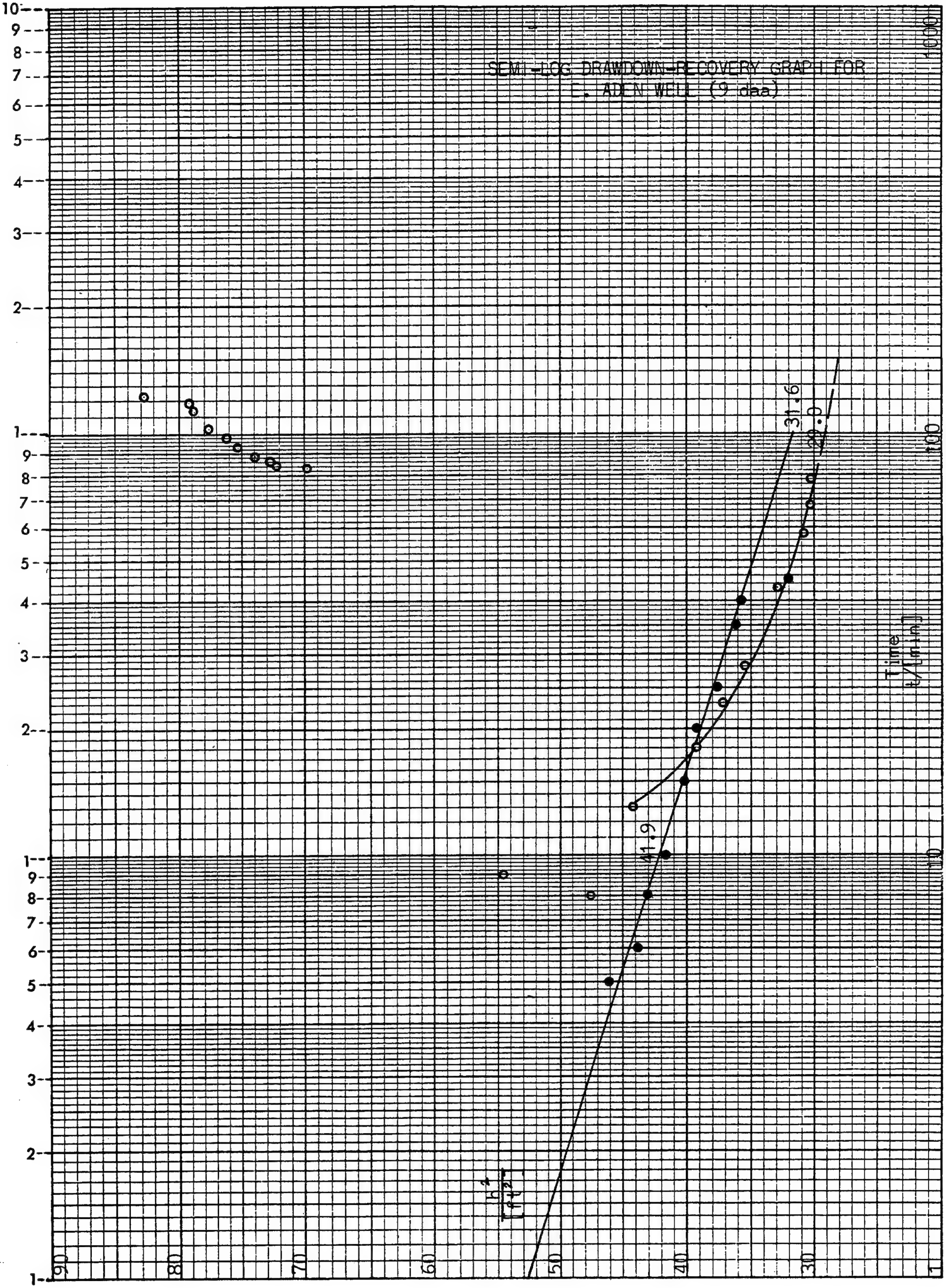
Figure D-3 is a semi-log drawdown-recovery graph for the Aden Well, on which are plotted values of the square of the depth of flow as observed in the well against the

FIG. D-2



17 September 1960

FIG. D-3


SEMI-LOG DRAWDOWN-RECOVERY GRAPH FOR
E. ADEN WELL (9 daa)
31.6
29.0
41.9
Time
t/[min]
1000
100
10
1
90
80
70
60
50
40
30

logarithm of the elapsed time. The square of the head difference is seen to drop from 41.9 ft^2 to 31.6 ft^2 between 10 and 100 minutes after start of the recovery. Thus the change in h^2 over one log cycle is 10.3 ft^2. (See column [11] of Table D-1.)

According to our approximate theory of radial flow in an unconfined aquifer, developed by analogy to flow in a confined aquifer, the relationship between the square of the depth of flow, the effective radius of the well, the elapsed time, and the formation constants is as follows:

$$(h_w^2) = (2.30\ Q/2\pi K)\log(2.25\ Kh^*t/S^*r_w^2)$$

From the change in h^2 over one log-cycle on the semi-log drawdown-recovery graph the hydraulic conductivity is determined from the following relationship which may be derived from the foregoing equation:

$$K = 2.30\ Q\ \Delta(\log t)/2\pi\Delta(h^2)$$

In this equation the subscript w has been omitted for convenience, as it has also on Figure D-3 and the following figures.

The measured discharge of the Aden well was 37,400 ft^3/day. The hydraulic conductivity is calculated to be 1,330 ft/day. (See columns [10] and [12], Table D-1.)

As it is not possible to determine the effective average storativity from a drawdown-recovery test, and as it is not possible either to determine the effective radius of the well, these two quantities are lumped together. Moreover as the effective average depth of flow is not known, it is here designated h* and combined with the other two unknowns. The logarithm of the quantity ($S^*\ r_w^2/h^*$ [ft]) may be determined from the drawdown observed in the well at any particular time, the hydraulic conductivity already having been found. For convenience we select the drawdown at 100 minutes, tabulated in column [15] of Table D-1. The quantity we seek, then, is given by the following formula:

$$\log(S^*r_w^2/h^*[ft]) = \log(2.25\ Kt/[ft]) - \Delta(h^2)/\ \Delta_1(h^2)$$

In this equation $\Delta_1(h^2)$ means the change of drawdown over one log cycle on the semi-log time drawdown graph. In the case of the Aden well this was 10.3 feet (Column [11]). Referring to Figure D-3, the drawdown, in feet squared, after 100 minutes of continuous pumping was (86.1 - 29.0) ft^2. As the ratio given in Column [14] is 5.55, the logarithm of the desired quantity, in Column [15], is 2.32 - 5.55 = -3.23 or 0.77 - 4.

The initial depth of flow at the Aden well is 9.3 feet, as given in Column [16] of Table D-1.

<u>Test of Dobbs Well</u>. Figure D-4 is a graph of observed depth to water during the drawdown-recovery test of the Dobbs well near Rocky Mountain Arsenal. Pertinent data on this

FIG. D-4

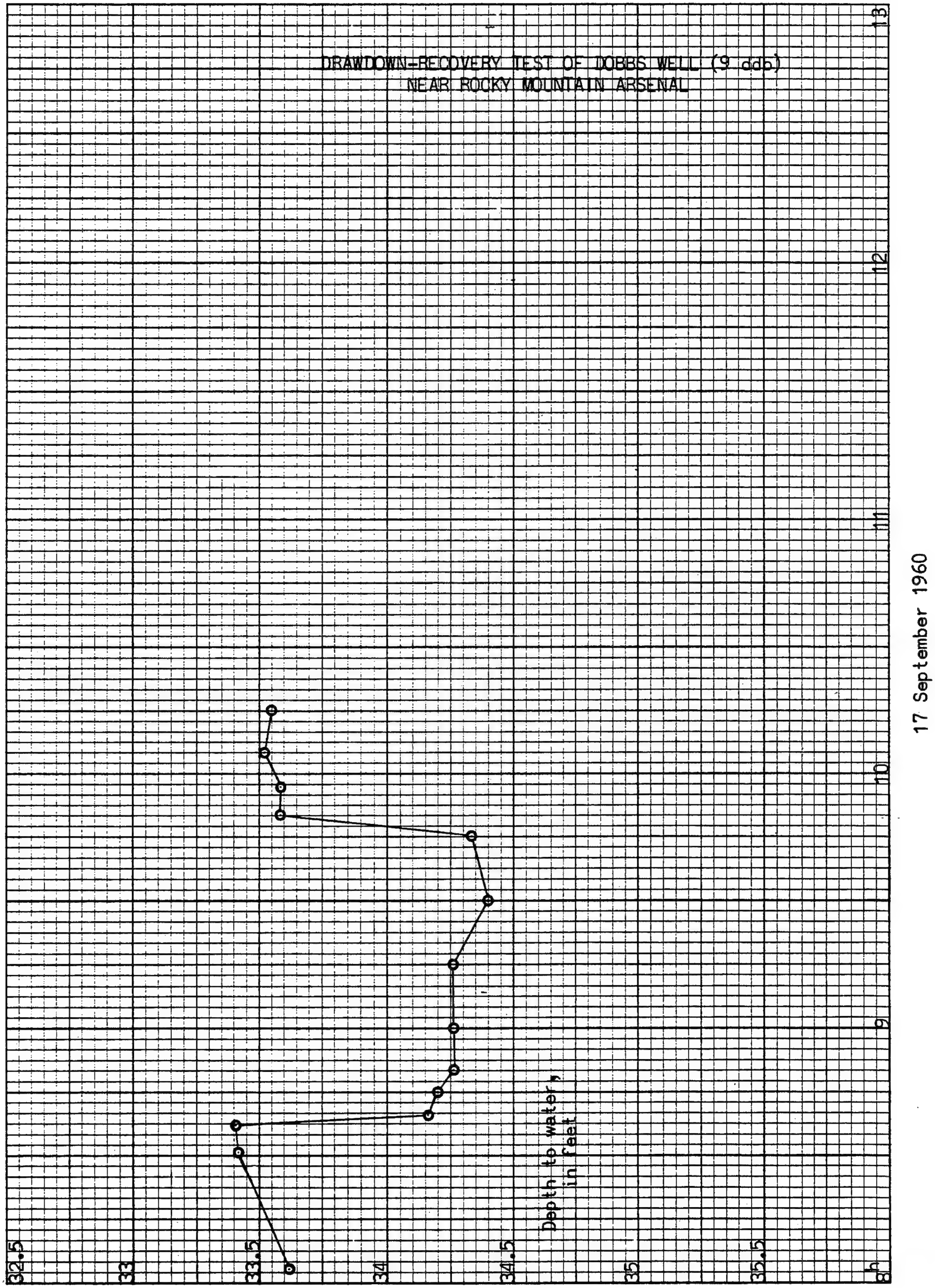
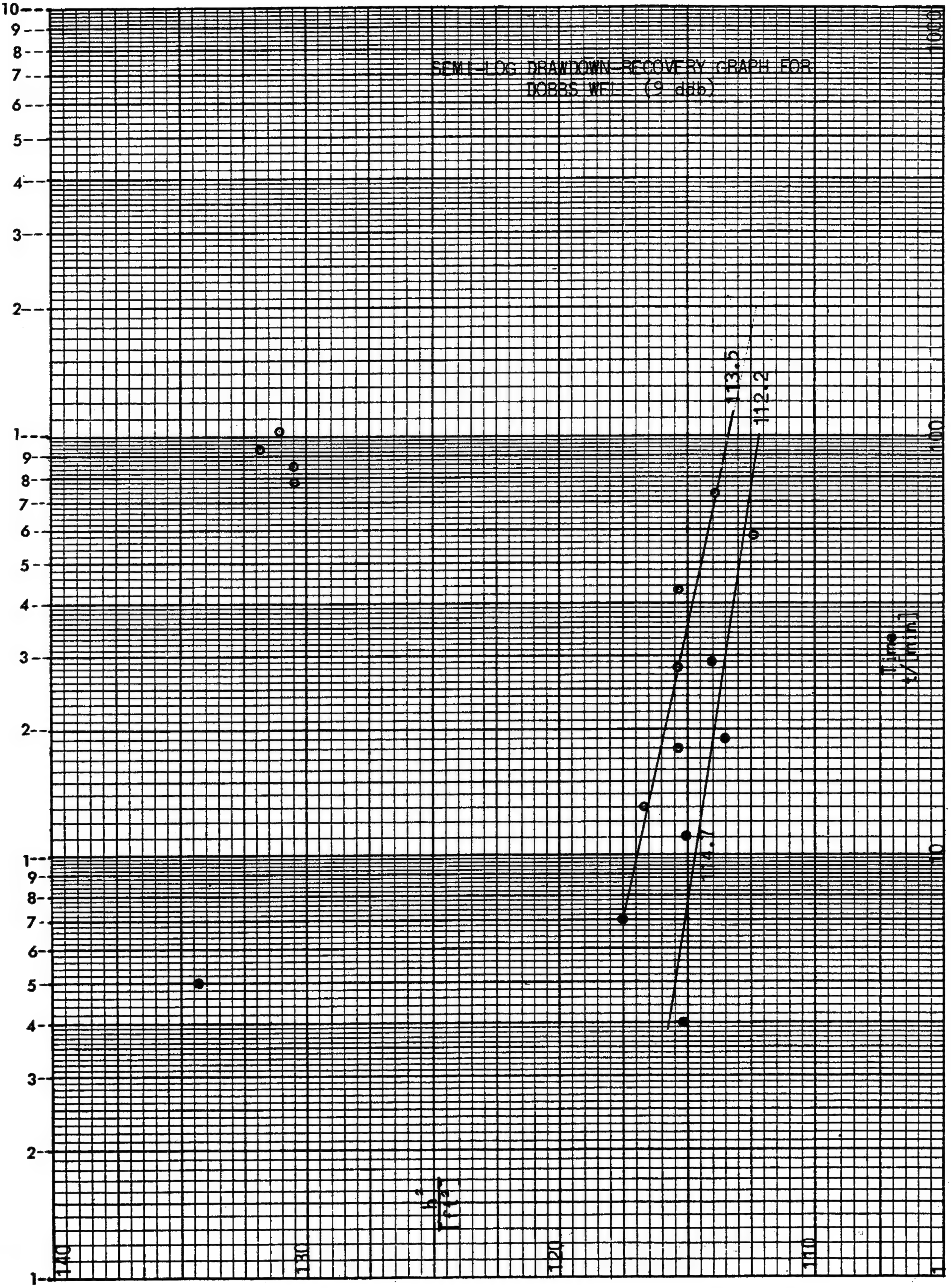

DRAWDOWN-RECOVERY TEST OF DOBBS WELL (9 ddb)
NEAR ROCKY MOUNTAIN ARSENAL
Depth to water, in feet
32.5
33
33.5
34
34.5
35
35.5
h
8
9
10
11
12
13
17 September 1960

FIG. D-5


SEMI-LOG DRAWDOWN-RECOVERY GRAPH FOR
113.5
112.2
114.7
Time
t/min
1000
100
10
1
140
130
120
110

test are as follows:

Date of test 17 September 1960.
Started pumping 08:32.
Stopped pumping 09:46.
Measured recovery until 10:15.
Discharge varied from 25 to 42 and averaged 42 gpm.
Discharge measured by 3-inch orifice.
Drilled depth 40 feet.
Sounded depth 45 feet.
Measuring point 1.1 feet above ground.
Type of screen or perforated section steel.
Length of same 4 inches to 6 inches.
For other pertinent data see Table D-1, second line.
Notes: "Well not used for one year. Owner claimed discharge about 400 gpm. Changed to small orifice at 08:34."

Figure D-5 is a semi-log drawdown-recovery graph of the Dobbs well. The points plotted there as circles represent the drawdown and those plotted as dots, the recovery. The recovery was measured up from the extrapolated drawdown and referred to a new beginning in time, namely the time of shutdown. In an ideal situation this transformation should bring the two sets of data into superposition. The spread of the data is attributable to inhomogeneities in the formation and also to the variability of the storativity.

In this case the change of h^2 over one log-cycle is only about 2.5 ft^2 and the hydraulic conductivity about 1,190 feet per day. The log of $(S^*r_w{}^2/h^*[ft])$ is 0.94 - 6. (See Columns [12] to [15] of Table D-1.)

Test of Marty Well. Figure D-6 is a drawdown-recovery graph for the test of the Marty well near Rocky Mountain Arsenal. Observations of depths to water, in feet, are plotted there against the clock time. Pertinent data on this test are as follows:

Date of test 21 September 1960.
Started pumping 08:55.
Stopped pumping 10:00.
Measured recovery until 15:16.
Discharge declined from 886 to 732 and averaged 765 gpm.
Discharge measured by 5-inch orifice.
Drilled depth 52 feet.
Sounded depth not given.
Measuring point flush with ground surface.
Type of screen or perforated section concrete.

FIG. D-6

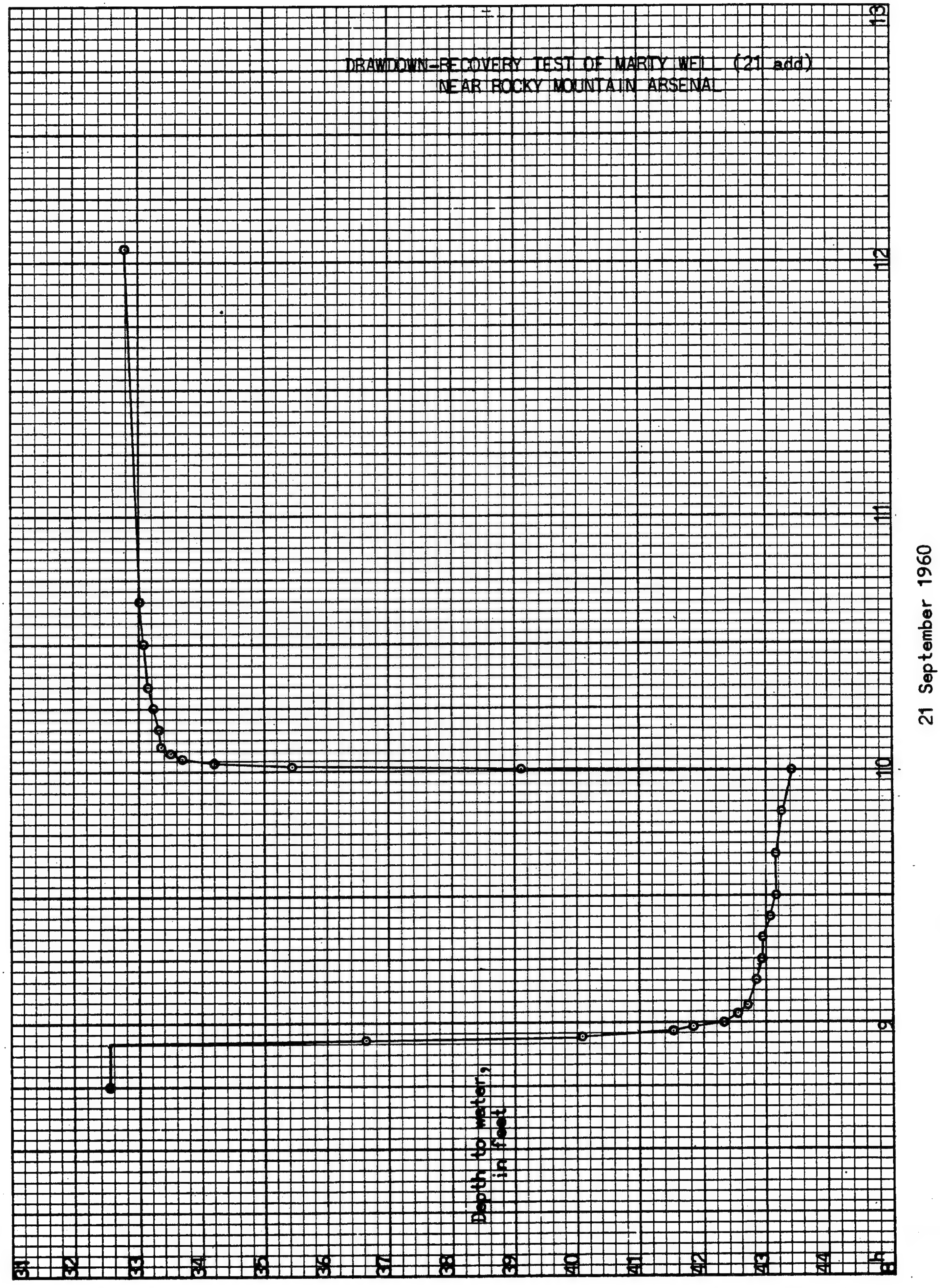
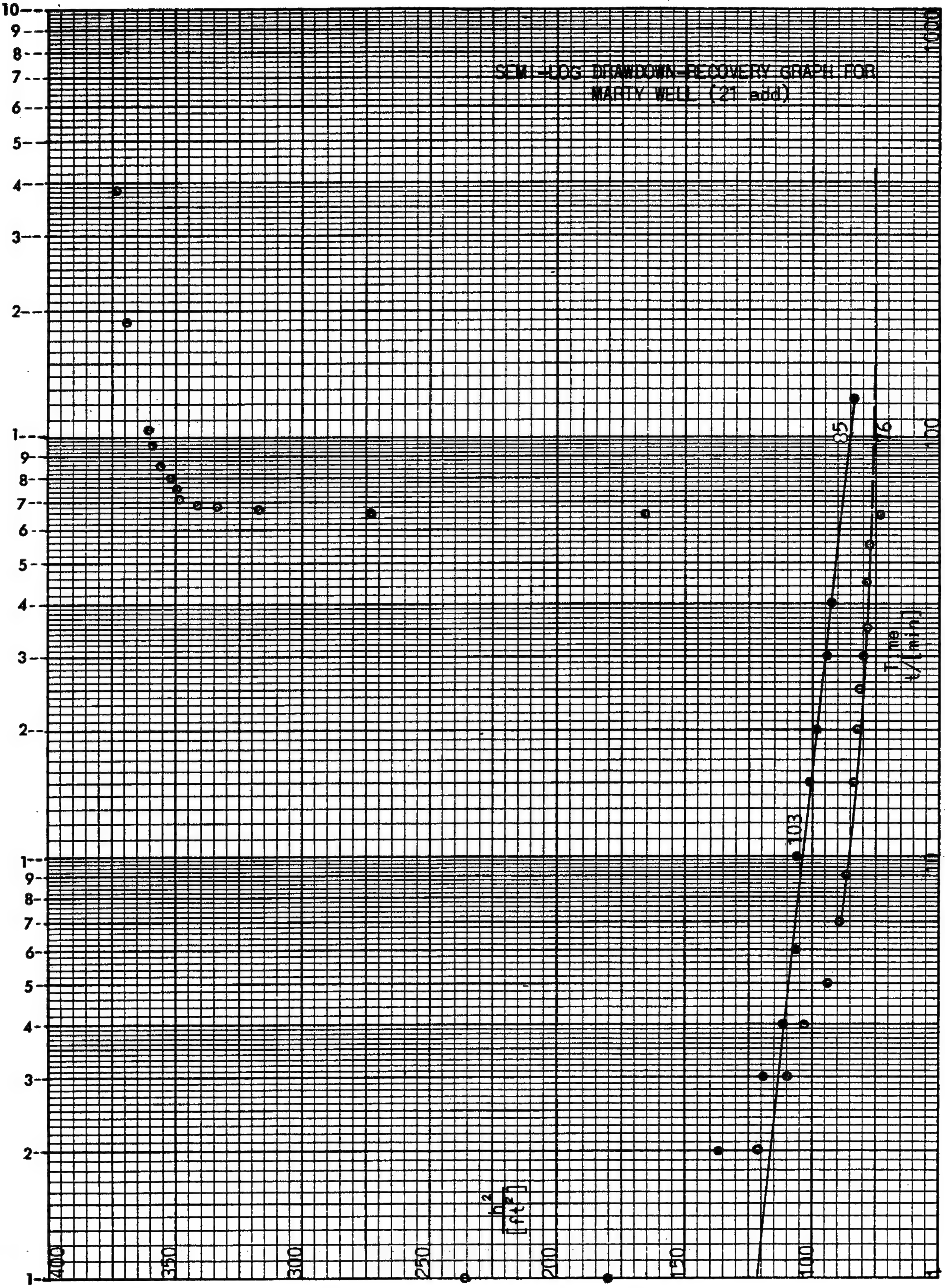

DRAWDOWN-RECOVERY TEST OF MARTY WELL (21 add)
NEAR ROCKY MOUNTAIN ARSENAL
Depth to water, in feet
21 September 1960
31
32
33
34
35
36
37
38
39
40
41
42
43
44
8
9
10
11
12
13

FIG. D-7


SEMI-LOG DRAWDOWN-RECOVERY GRAPH FOR
MARTY WELL (21 add)
h² [ft²]
Time
t/[min]
400
350
300
250
200
150
100
1000
100
10
1
85
76
103

Length of well screen not known.

For other pertinent data see Table D-1, third line.

Figure E-7 is a semi-log drawdown-recovery graph for the Marty well. The circles represent calculated values of h^2 determined from observations of depth to water during the pumping and subsequent recovery. The dots represent the recovery replotted on the logarithmic time scale, taking the time of shutdown as the new origin.

The change of h^2 over one log-cycle is 18 feet, and the hydraulic conductivity is calculated to be about 3,000 ft/day. The "drawdown" at 100 minutes is 377 - 76 or 301 ft^2. Accordingly the log of $(S^* r_w{}^2/h^2[ft]) = .97 - 15$. The reason for the high negative value here is that the well is strong and the drawdown is large in comparison to the initial depth of flow. This entails high convergence losses as the water decends toward the well. The initial depth of flow in this case was 19.4 feet.

Test of Masunaga Well. Figure D-8 is a drawdown-recovery graph of the test of the Masunaga well near Rocky Mountain Arsenal. The well was first pumped for about 4 minutes starting at 12:05, when it was found necessary to reduce the size of the orifice. After waiting several minutes, pumping was resumed. It is believed that this short initial pumping period after the false start had little effect on the subsequent test and accordingly corrections were not made for it. Pertinent data on this test are as follows:

Date of test 20 September 1960.

Started pumping 12:41.

Stopped pumping 13:40.

Measured recovery until 14:15. (Two measurements were made the following day.)

Discharge declined from 183 to 169 and averaged 176 gpm.

Discharge measured by 3-inch orifice.

Drilled depth 39 feet.

Sounded depth not given.

Measuring point 0.9 foot above ground.

Type of screen or perforated section, concrete.

Length of same, 12 feet (?).

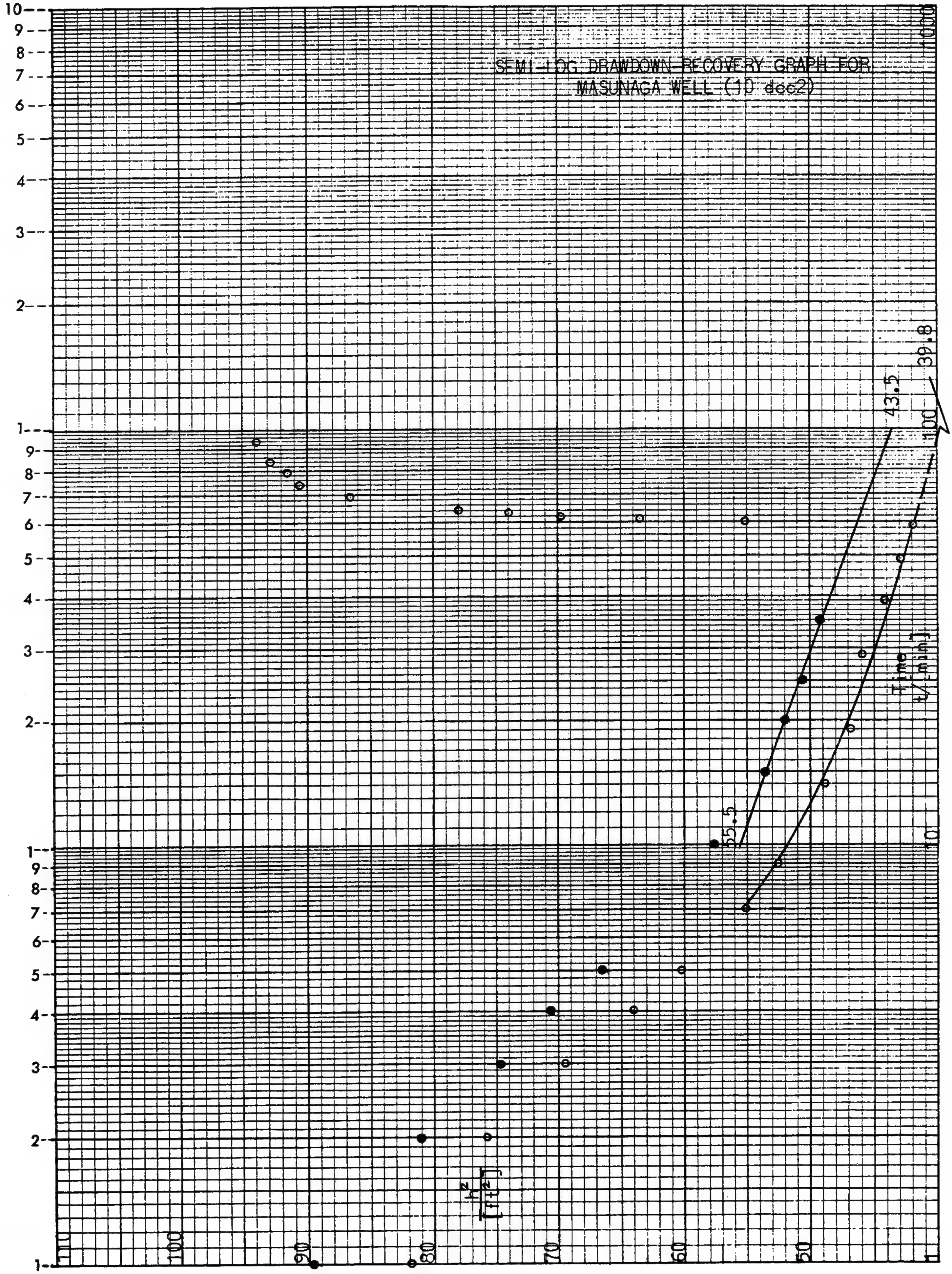
For other pertinent data see Table D-1, fourth line.

Notes: "In this test we observed the well pumped and a well which was [some distance] away. The well reading in the remarks column [of the data sheet] has a correction of +0.25 ft to each reading. Well 1355 [Masunaga Well] is connected to two wells by siphons. When we pumped, we took readings on Well 1355 and Well A. I was not aware of Well B until pumping had been completed."

FIG. D-8


DRAWDOWN-RECOVERY TEST OF MASUNAGA WELL (10 dcad)
NEAR ROCKY MOUNTAIN ARSENAL
Depth to water, in feet
27
28
29
30
31
32
33
10h
11
12
13
14
15
20 September 1960

FIG. D-9



SEMI-LOG DRAWDOWN-RECOVERY GRAPH FOR MASUNAGA WELL (10 dec2)
43.5
39.8
55.5
Time t/[min]
h/[ft]

Figure D-9 is a semi-log drawdown-recovery graph for the Masunaga Well . Again the circles represent calculated values of h^2 plotted against the logarithm of time during the drawdown and subsequent recovery. The dots represent the recovery referred back to the new time origin and expressed as a differential of h^2 measured up from the extrapolated drawdown curve.

The change of h^2 over one log cycle is in this case 12 feet, and the hydraulic conductivity is calculated to be 1,040 ft/day. The drawdown at 100 minutes was 102.8 - 39.8 or about 63 ft^2. The log of $(S^*r_w{}^2/h^*[ft])$ is 0.96 - 4. In this case the initial depth of flow was only 10.1 ft.

<u>Test of Matsumoto Well</u>. Figure D-10 is a drawdown-recovery graph for the test of the Matsumoto well near Rocky Mountain Arsenal. Pertinent data on this test are as follows:

Date of test 22 September 1960.
Started pumping 08:00 .
Stopped pumping 09:05.
Measured recovery until 13:35. (One measurement following morning.)
Discharge declined from 357 to 326 and averaged 336 gpm.
Discharge measured by 4-inch orifice.
Drilled depth not given.
Sounded depth 36.9 feet.
Measuring point 2.75 feet above ground.
Type of screen or perforated section, concrete.
Length of same, 6 feet.
For other pertinent data see Table D-1, fifth line.

Figure D-11 is a semi-log drawdown-recovery graph for the Matsumoto well. The change of h^2 over one log-cycle is in this case 9.0 ft, and the hydraulic conductivity is calculated to be 2,640 ft/day. The drawdown at 100 minutes, expressed in terms of h^2 is 131.8 - 44.0, or 87.8 ft^2. The log of $(S^*r_w{}^2/h^*[ft])$ is 0.85 - 8. In this case the initial depth of flow was 11.5 feet.

<u>Test of Matsumoto Well 2.</u> Figure D-12 is a drawdown-recovery graph for the test of Matsumoto Well 2 near Rocky Mountain Arsenal. Pertinent data for this test are as follows:

Date of test 22 September 1960.
Started pumping 11:55.
Stopped pumping 13:00.
Measured recovery until 13:50 (one measurement following morning).
Discharge declined from 447 to 368 and averaged 384 gpm.
Discharge measured by 4- inch orifice.
Drilled depth 45 feet.

FIG. D-10

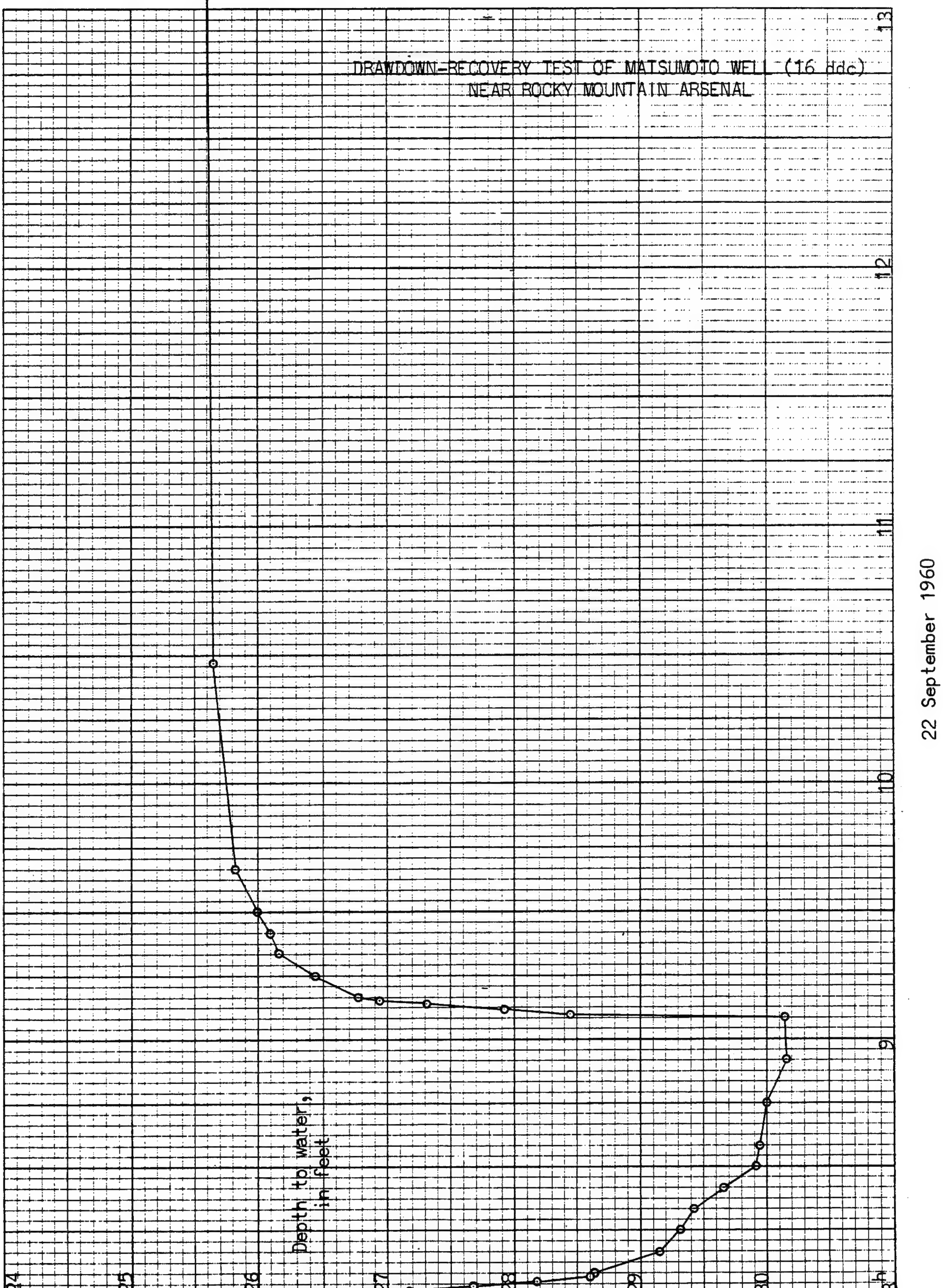

DRAWDOWN-RECOVERY TEST OF MATSUMOTO WELL (16 ddc)
NEAR ROCKY MOUNTAIN ARSENAL
Depth to water, in feet
24
25
26
27
28
29
30
8h
9
10
11
12
13
22 September 1960

FIG. D-11

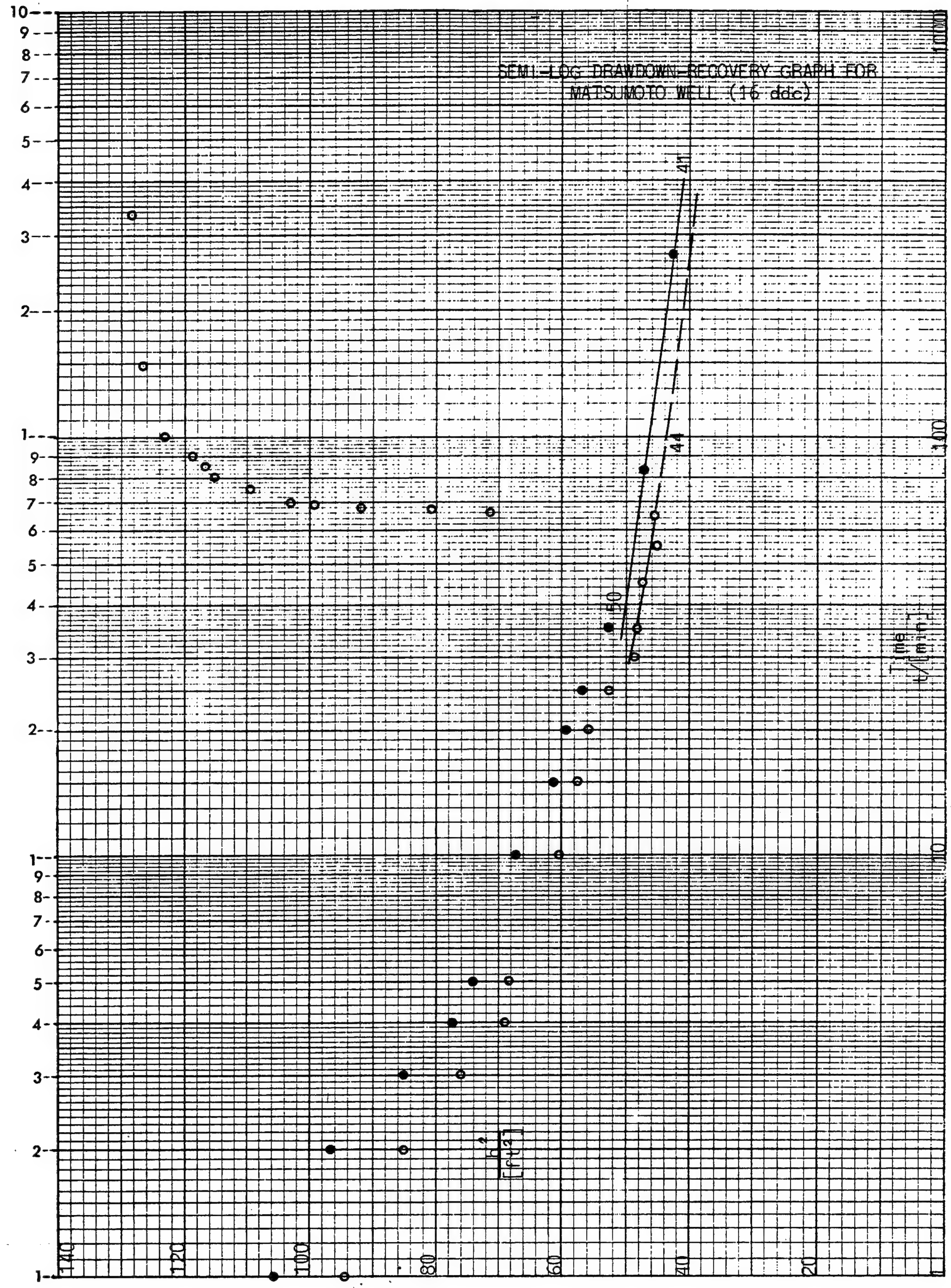
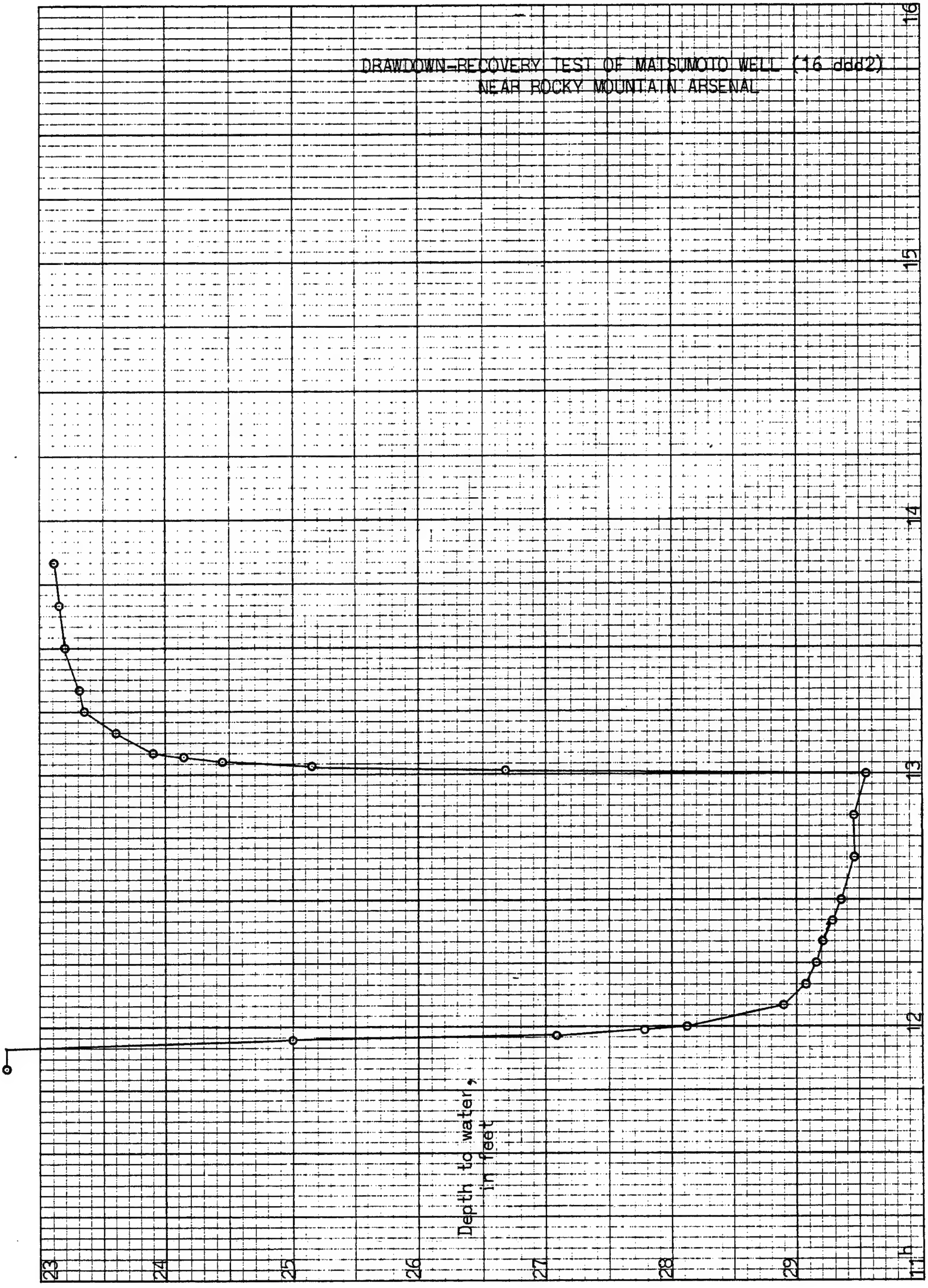
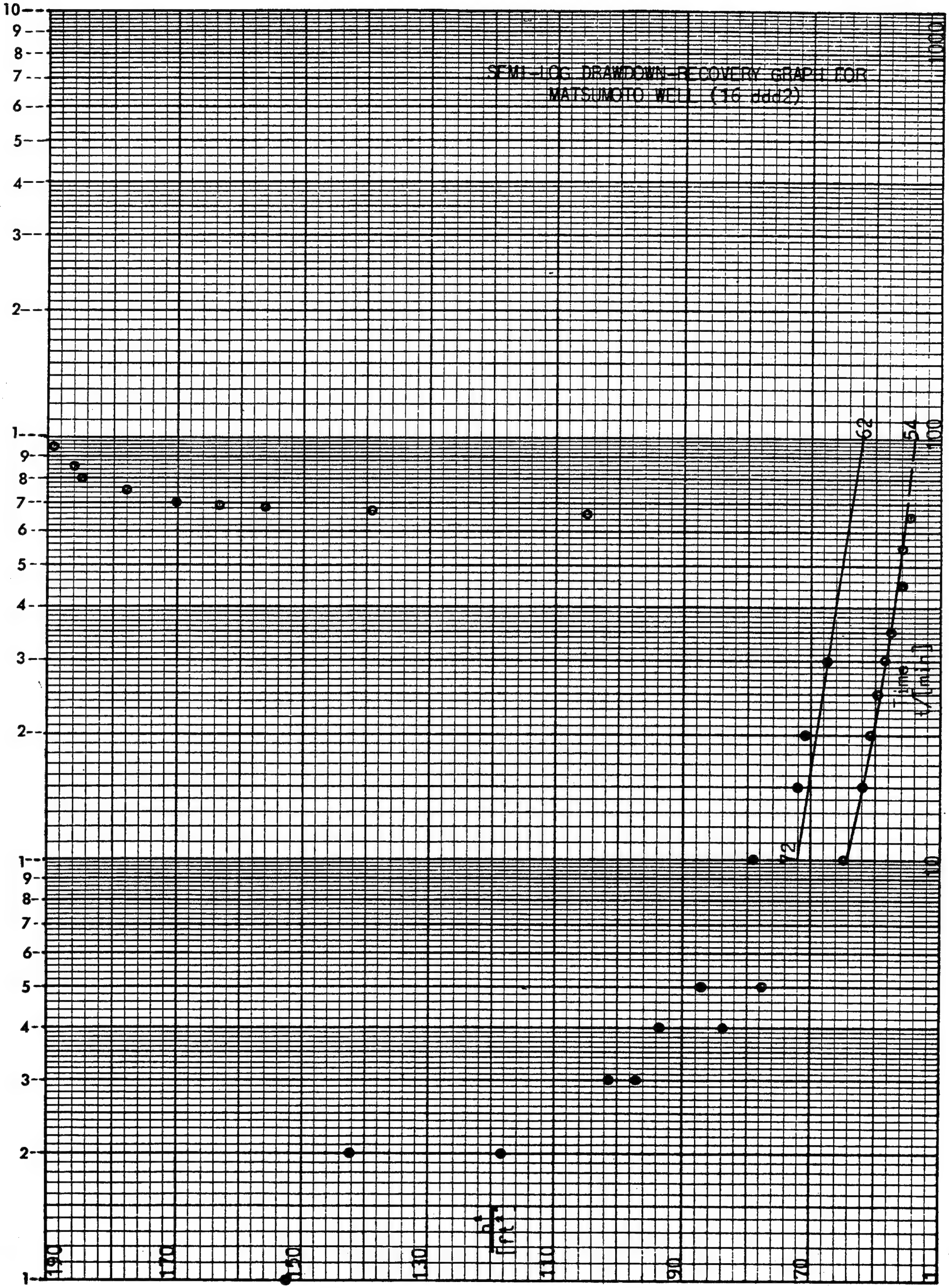

SEMI-LOG DRAWDOWN-RECOVERY GRAPH FOR
MATSUMOTO WELL (16 ddc)
1000
100
10
1
t/[time/min]
h²[ft²]
140
120
100
80
60
40
20
50
41
44

FIG. D-12



22 September 1960

FIG. D-13


SEMI-LOG DRAWDOWN-RECOVERY GRAPH FOR
MATSUMOTO WELL (16 ddd2)
62
54
72
t/t' (min)
h (ft)
190
170
150
130
110
90
70
1
10
100
1000

Sounded depth 36.9 feet.
Measuring point 1.0 feet above ground.
Type of screen or perforated section, concrete.
Lenth of same, 6 feet.
For other pertinent data see Table D-1, sixth line.

Figure D-13 is a semi-log drawdown-recovery graph for Matsumoto Well 2. The same convention is used as before in the plotted points. The change of h^2 over one log-cycle is 10 ft, and the hydraulic conductivity is calculated to be 2,720 ft/day. The drawdown at 100 minutes expressed in terms of h^2 is 201.9 - 54.0, or 147.9 ft^2. The log of $(S^*r_w{}^2/h^*[ft])$ is 0.83 - 13. The reason for the high negative logarithm here is that the well is strong and the drawdown a larger fraction of the initial depth of flow than ordinarily. In this case the initial depth of flow was 14.2 feet.

Test of Monson Well. Figure D-14 is a drawdown-recovery graph for the test of the Monson well near Rocky Mountain Arsenal. Pertinent data for this test are as follows:

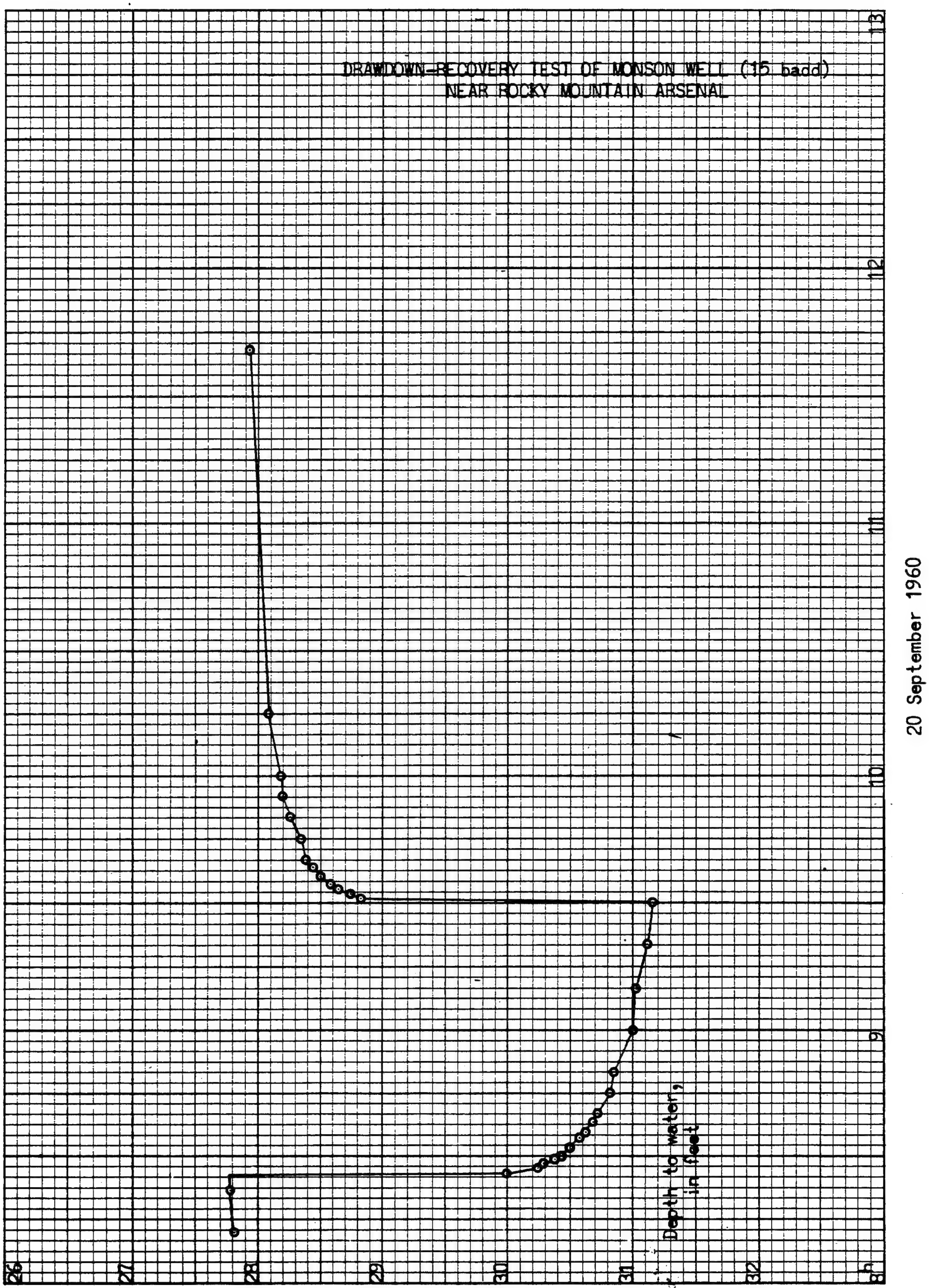
Date of test 20 September 1960.
Started pumping 08:25.
Stopped pumping 09:30.
Measured recovery until 11:41 (one measurement afternoon of following day).
Discharge declined from 400 to 377 and averaged 386 gpm.
Discharge measured by 4-inch orifice.
Drilled depth 40 feet, plus or minus.
Sounded depth not given.
Measuring point 1.7 feet above ground.
Type of screen not given.
Lenth of same, not given.
For other pertinent data see Table D-1, seventh line.
Notes: "Well in operation morning of 19 September 1960. Owners' well logs have been sent to State Engineer's office."

Figure D-15 is a semi-log drawdown-recovery graph for the Monson well. The change of h^2 over one log-cycle is 12.5 feet, and the hydraulic conductivity is calculated to be 2,180 ft/day. The drawdown at 100 minutes is 148.9 - 76.0, or about 72.9 ft^2. The log of $(S^*r_w{}^2/h^*[ft])$ is 0.70 - 4. In this case the initial depth of flow was 12.2 feet.

Test of Myers Well. Figure D-16 is a drawdown-recovery graph for the test of the Myers well near Rocky Mountain Arsenal. Pertinent data on this test are as follows:

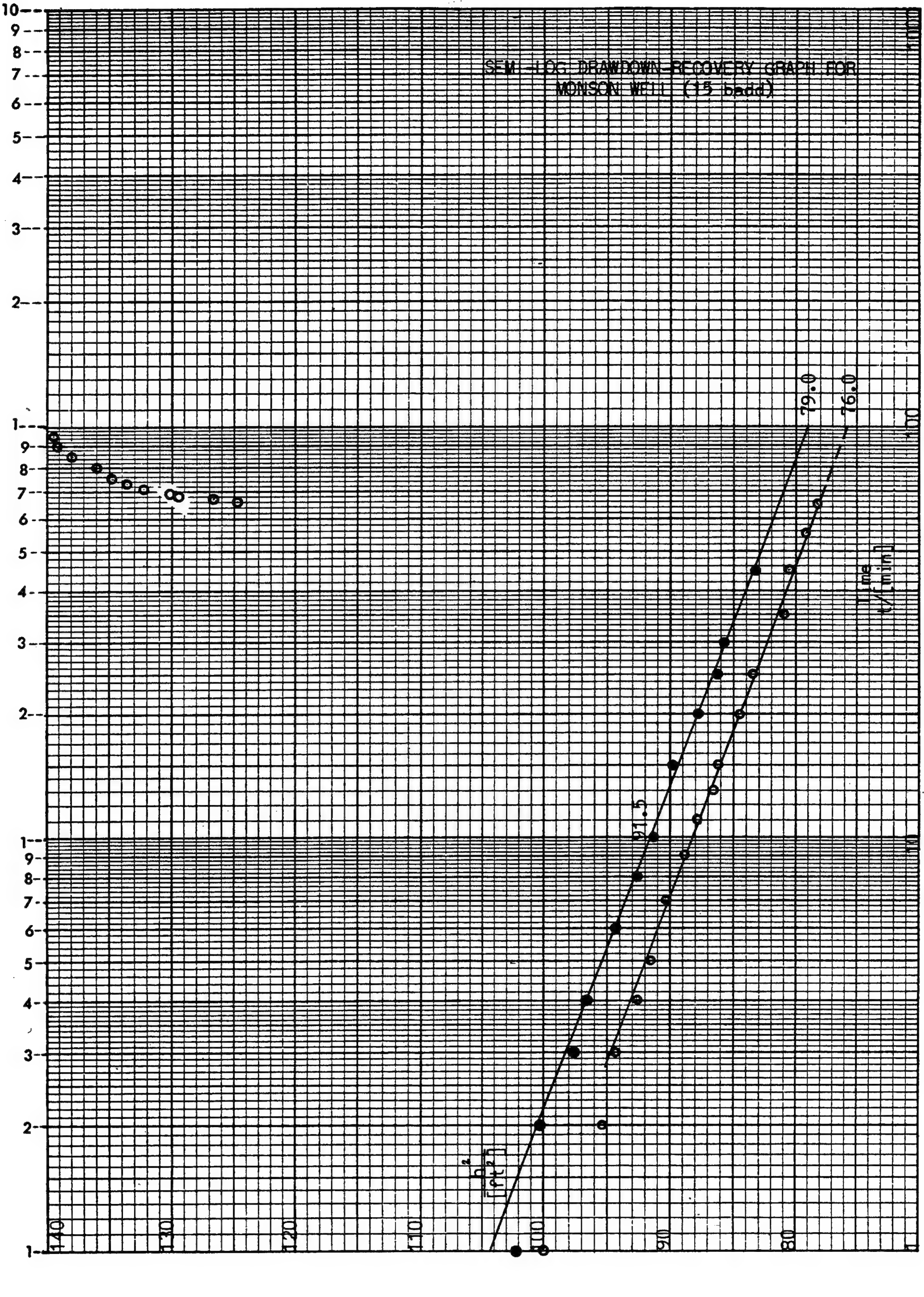
Date of test 21 September 1960.
Started pumping 13:15.

FIG. D-14



DRAWDOWN-RECOVERY TEST OF MONSON WELL (15 badd)
NEAR ROCKY MOUNTAIN ARSENAL
Depth to water, in feet
20 September 1960
26
27
28
29
30
31
32
9
10
11
12
13

FIG. D-15



SEMI-LOG DRAWDOWN-RECOVERY GRAPH FOR
MONSON WELL (15 badd)
79.0
76.0
91.5
Time
t/ [min]
h^2
[ft²]
140
130
120
110
100
90
80

Stopped pumping 14:10.
Measured recovery until 14:50.
Discharge declined from 54 to 42 and averaged 46 gpm.
Discharge measured by 3-inch orifice.
Drilled depth not given.
Sounded depth 39.5 feet.
Measuring point flush with ground surface.
Type of screen not given.
Length of screen, 4 feet (?).
For other pertinent data see Table D-1, eighth line.
Notes: "The discharge figures are estimated. I plotted head against drawdown and extrapolated to zero."

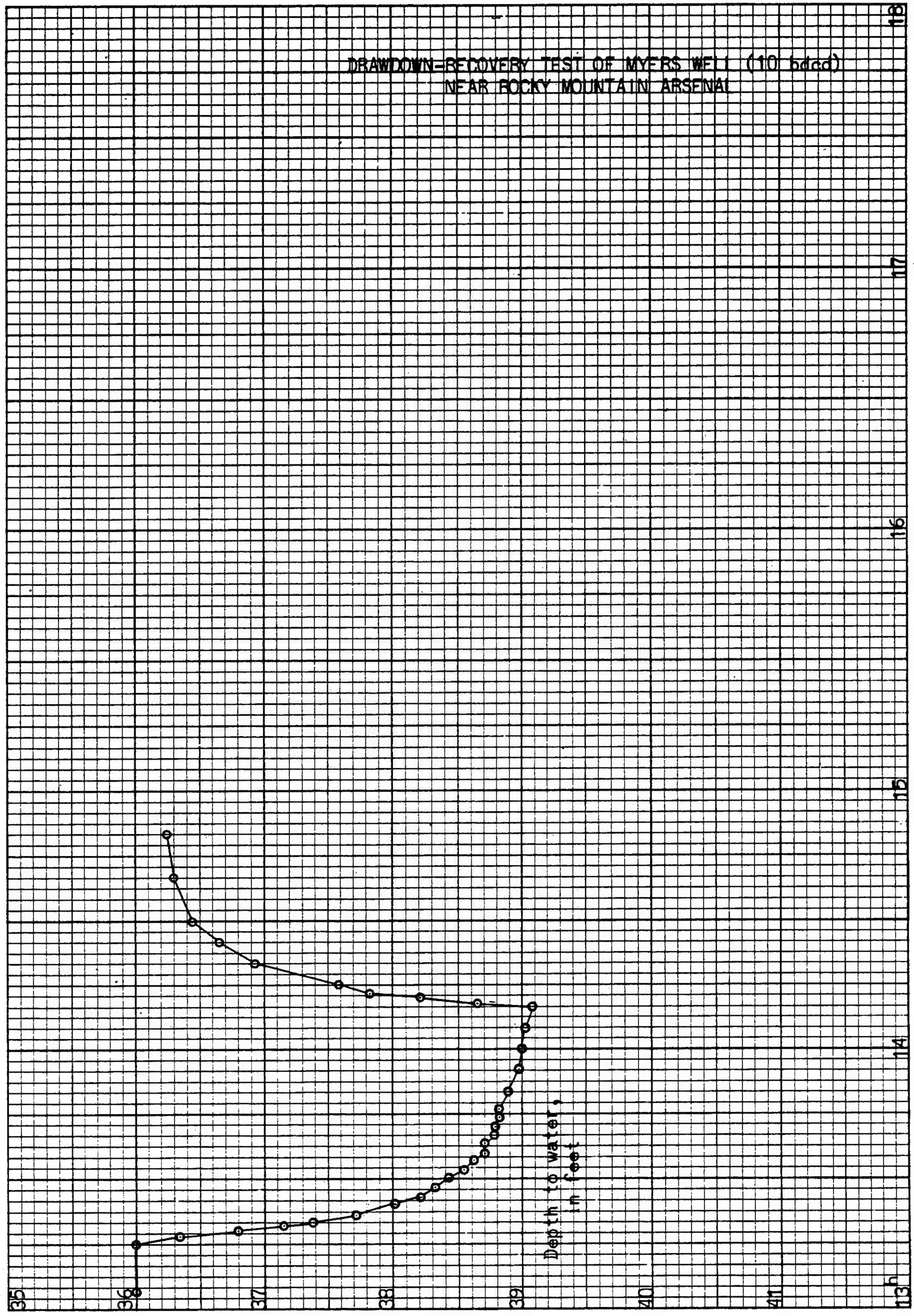
Figure D-17 is a semi-log drawdown-recovery graph for the Myers well. The change of h^2 over one log-cycle is less than 2.5 ft, and the hydraulic conductivity is calculated to be more than 1,300 ft/day. The drawdown at 100 minutes is 12.25 + .05, or about 12.3 ft^2. The log of ($S^*r_w^2/h^*[ft]$) is 0.41 - 3. In this case the initial depth of flow was only 3.5 ft.

Test of Wolpert Well. Figure D-18 is a drawdown-recovery graph for the test of the Wolpert well near Rocky Mountain Arsenal. Pertinent data for this test are as follows:

Date of test 23 September 1960.
Started pumping 08:25.
Stopped pumping 09:31.
Measured recovery until 10:30.
Discharge declined from 282 to 274 and averaged 275 gpm. (Initial discharge 262 gpm.)
Discharge measured by 5-inch and 3-inch orifices.
Drilled depth not given.
Sounded depth 18.1 feet.
Measuring point flush with ground surface.
Type of screen of perforated section, concrete.
Lenth of same, not given.
For other pertinent data see Table D-1, tenth line.
Note: "Loss 1 gpm."

Summary of Tests. Table D-1 gives a summary of the ten drawdown-recovery tests run on wells near Rocky Mountain Arsenal. Included among these ten is the Powers well, which was given a step-drawdown test. It is reported on more fully in the next section of this Appendix.

FIG. D-16



21..September 1960

FIG. D-17

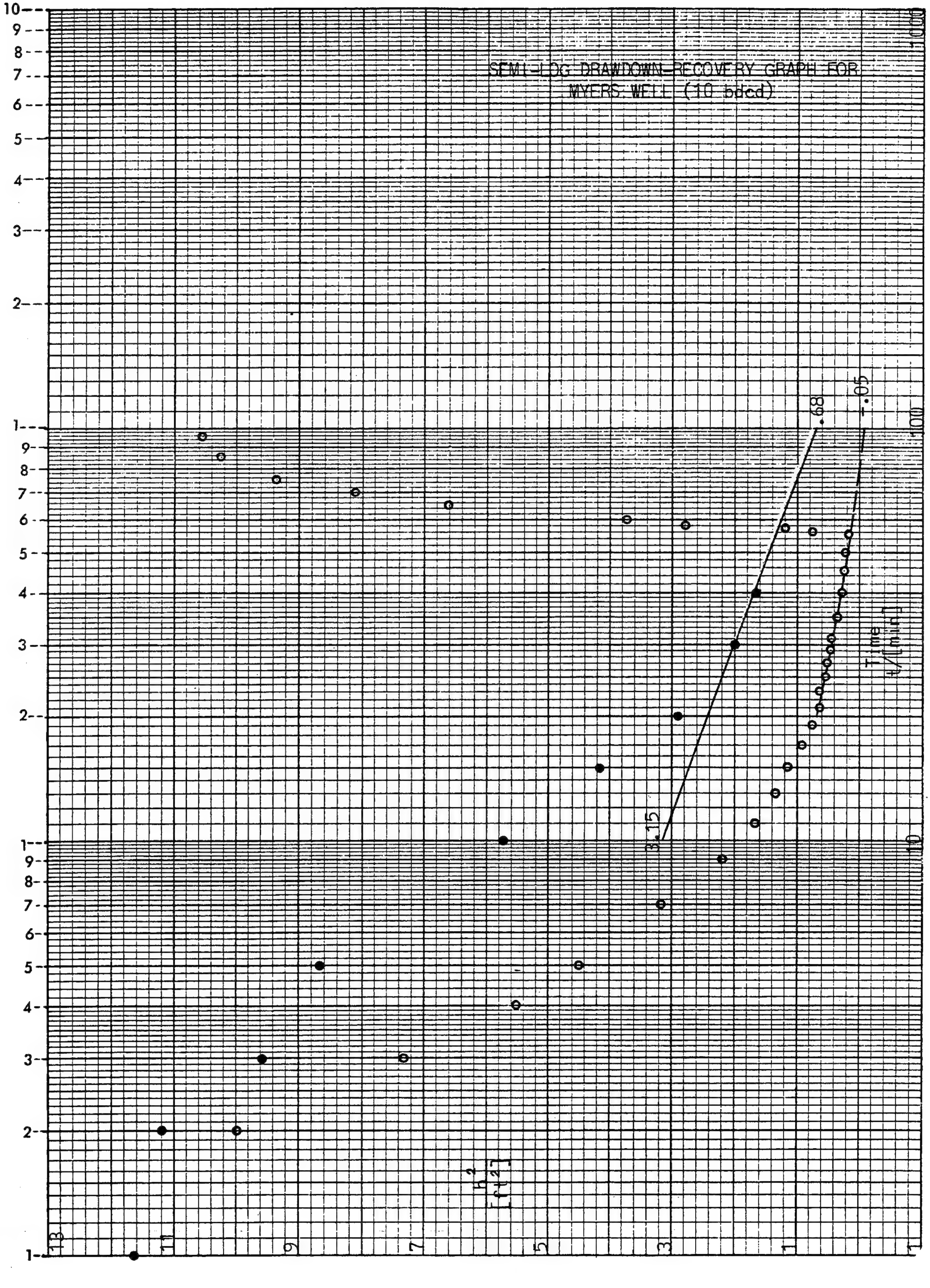

SEMI-LOG DRAWDOWN-RECOVERY GRAPH FOR
MYERS WELL (10 bdcd)
.68
.05
3.15
Time
t/(min)
h²
(ft²)
1000
100
10
1
13
11
9
7
5
3
1
10
9
8
7
6
5
4
3
2
1

FIG. D-18

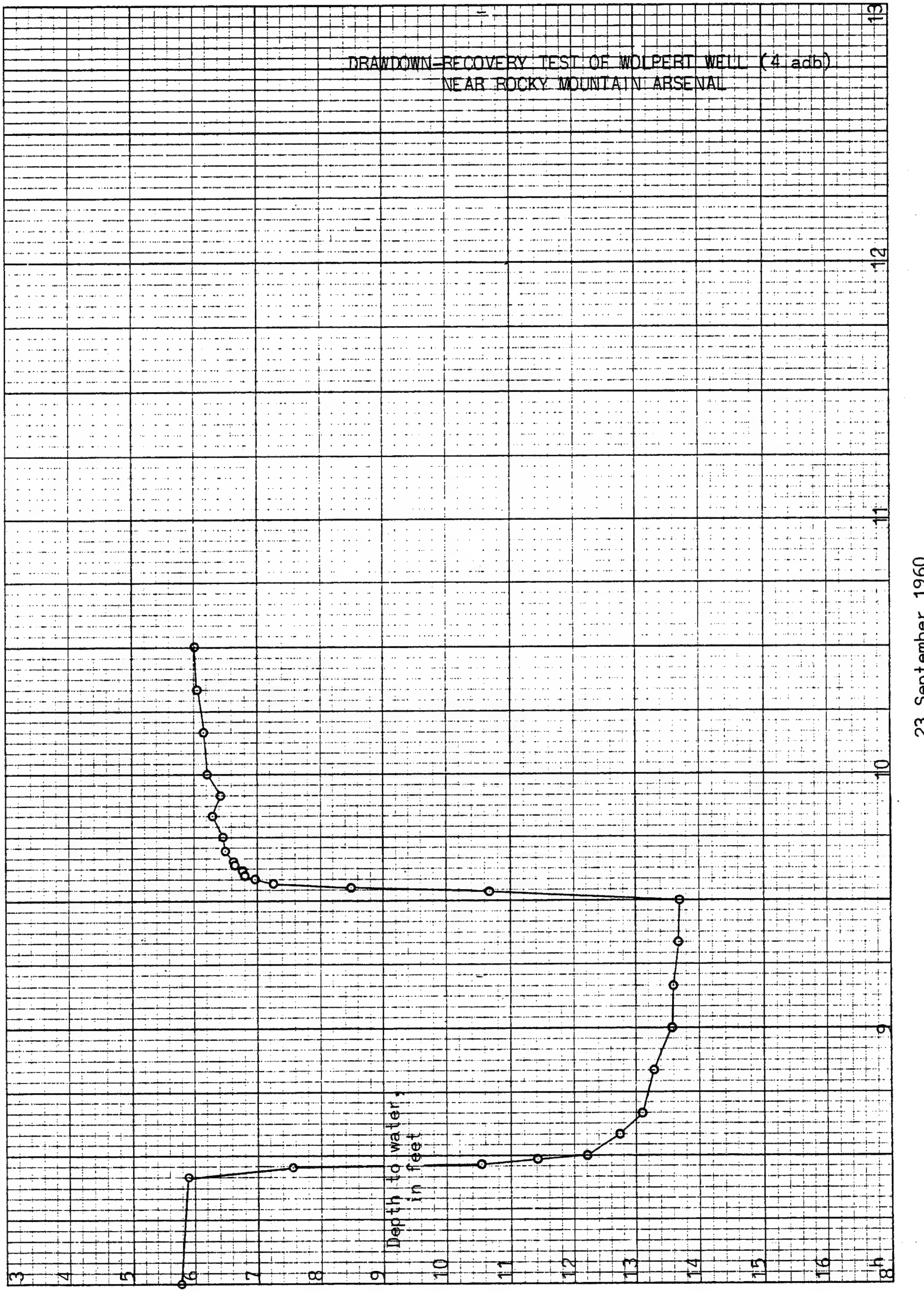
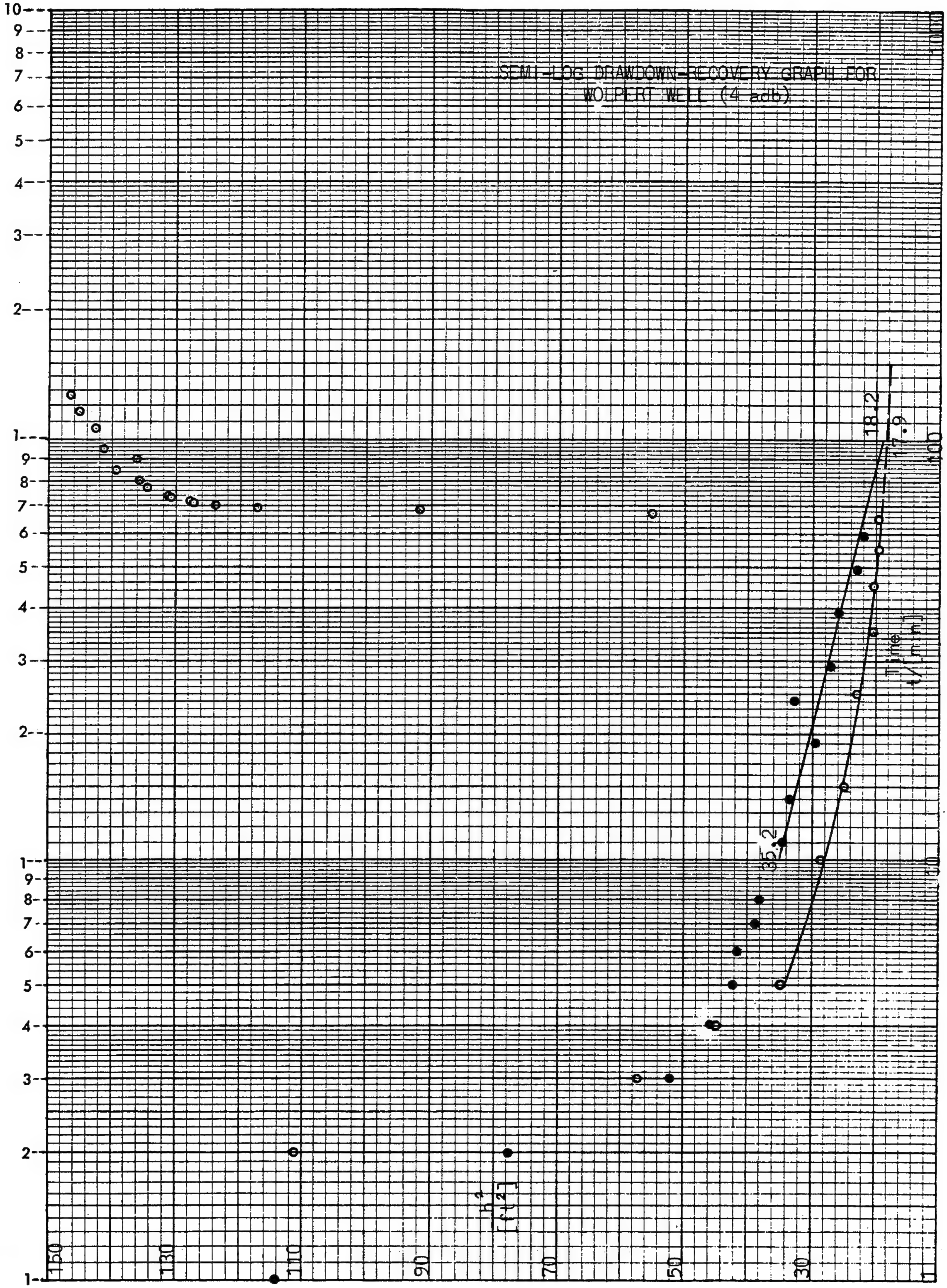

DRAWDOWN-RECOVERY TEST OF WOLPERT WELL (4 adb)
NEAR ROCKY MOUNTAIN ARSENAL
Depth to water, in feet
23 September 1960
3
4
5
6
7
8
9
10
11
12
13
14
15
16
8
9
10
11
12
13

FIG. D-19


SEMI-LOG DRAWDOWN-RECOVERY GRAPH FOR
WOLPERT WELL (4 adb)
18.2
7.9
35.2
Time
t/[min]
h²
[ft²]
150
130
110
90
70
50
30
1
10
100
10
9
8
7
6
5
4
3
2
1

Column [1] of Table D-1 gives the name of the owner of the well and column [2] the location by section number and fraction thereof within T2S R67W. Column [3] gives the depth of the well, in feet. Those depths shown to the nearest foot are drilled depths, while those shown to tenths or hundreths are depths that were sounded during the tests.

Columns [4] and [5] give the diameter of the casing in inches and the kind of material of which constructed, respectively. Columns [6] and [7] give the diameter of the pump in inches and its power in horsepower, respectively.

In inventorying the wells each owner was asked the estimated discharge of his well. In every case but one the owners overestimated the discharge, sometimes by two or three times. (Compare Columns [8] and [9] of Table D-2.)

The discharge measured by pipe orifice during the test is expressed in gpm in Column [9] and in thousands of ft^3/day in Column [10]. The figures in Columns [11] through [16] have already been discussed in the foregoing discussion of the individual tests.

The average depth of the ten wells tested is 37.2 feet. The most common casing diameter 48 inches and the most common material concrete. The average initial depth to flow is 11.6 feet. The average discharge 287 gpm, or 55,200 ft^3/day. These average figures are given at the bottom of the table. Also given there are averages calculated after excluding the two starred numbers in each column, namely those for the Marty well and the Matsumoto No. 2 well. These two wells are excluded because of their inordinately high discharge and strong drawdown in comparison to the initial depth of flow. The remaining eight wells have an average discharge of 215 gpm or 41,400 ft^3/day. For these eight tests the average hydraulic conductivity was about 1,500 ft/day. The average log of $(S^* r_w^2 / h^*[ft])$ for the eight wells is 0.72 - 5 and the average initial depth of flow 10.3 feet.

In order to calculate the future drawdown around similar wells it may not be necessary to separate out the effective average storativity, the effective average depth to flow, and the effective radius of the well. Because of the crudity of the theory there may be uncertainties both in S^* and in h^*. Variations in one or both may be reflected in unreal values of r_w^2 if attempts are made to calculate that magnitude. For our purpose, we may use 0.72 - 5 or -4.28 as the logarithm of this combined quantity in predicting the behavior of similar wells over long continued pumping.

Step-Drawdown Test of Powers Well

<u>Theory and procedure</u>. The theory and procedure of step-drawdown tests is given on page B-4 of Appendix B and on page 5 of Memorandum of Conference of 9 August 1960.

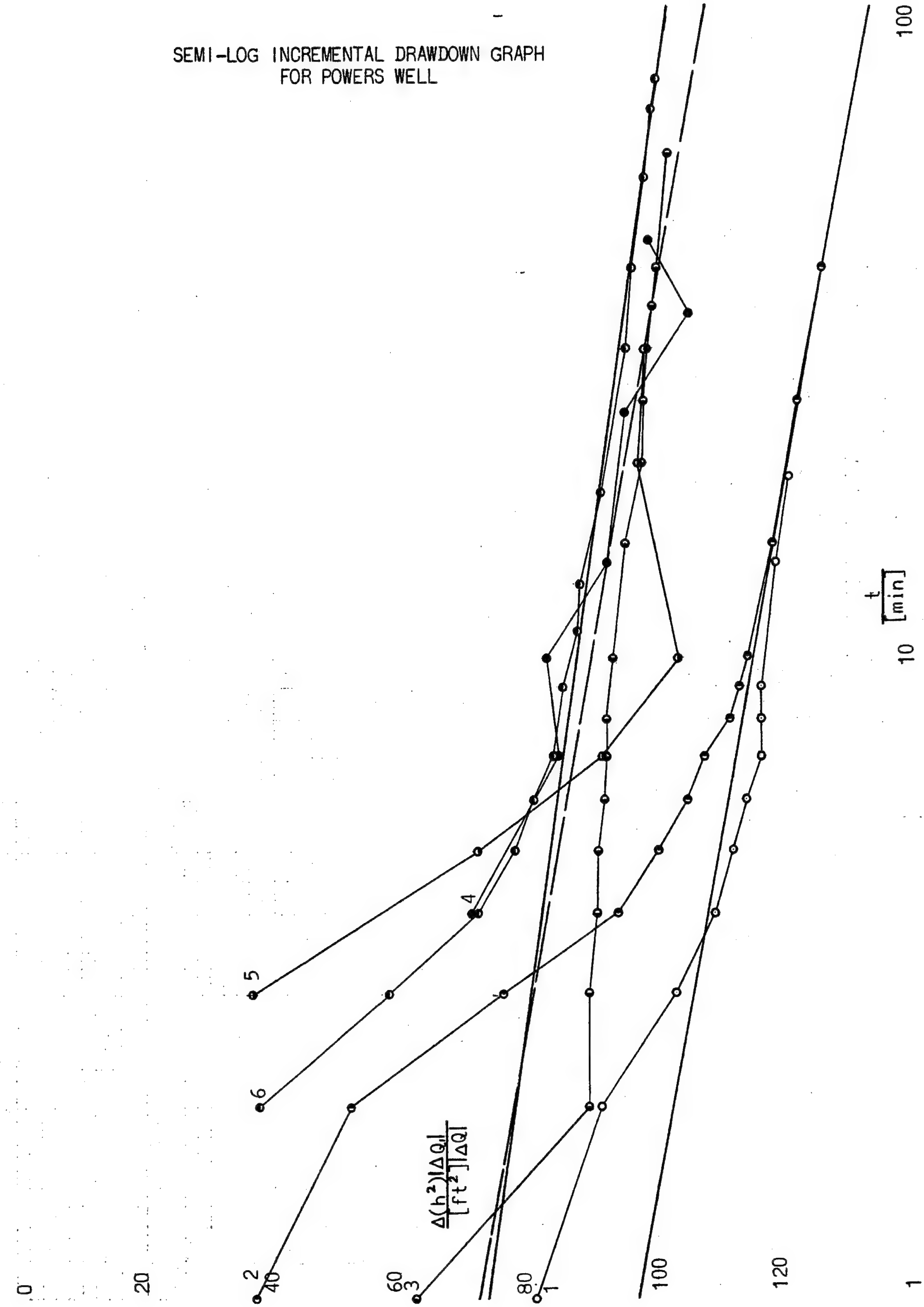
<u>Pertinent Data on test</u>. Figure D-20 is a graph of the step-drawdown test of the Powers well near Rocky Mountain Arsenal. At the bottom of the graph calculated values of h^2, in ft^2, are plotted against the clock time. At the top of the graph discharge readings, in gpm, are plotted on the same time scale. There are six steps altogether in the test. The well was started at full capacity and pumped for about 20 minutes. After being allowed to recover for about 45 minutes it was started up again at a fraction of full capacity, as indicated on the graph. Finally, after increasing the discharge in two subsequent steps, the well was shut off and allowed to fully recover. Pertinent data on this test are as follows:

Date of test 19 September 1960.
Started pumping 12:01.
Stopped pumping (final) 15:20.
Measured recovery until 16:38 (one measurement following morning).
Discharge varied from step to step.
Discharge measured by 5-inch orifice.
Drilled depth 40 feet.
Sounded depth not given.
Measuring point 1.1 feet above ground.
Type of screen or perforated section, concrete with 1/4-inch slots, V-shaped.
Length of same, 10 feet.
For other pertinent data see Table D-1, ninth line; also Table D-2.
Notes: "Well not used for four years. The owner had estimated 800 gpm +. We installed a 5-inch orifice. Pumped for 20 minutes, stopped pump, and allowed well to recover. Changed to 3-inch orifice and started pumping at 13:05 for a step-drawdown [test]. Had to stop pumping at 15:20, beginning to flood crops."

Figure D-21 is a semi-log incremental drawdown graph for the step-drawdown test of the Powers well. Plotted thereon are calculated values of the increment of h^2 divided by the corresponding increment of discharge, but multiplied by the increment of discharge during the first step in order to keep the dimensions [ft^2]. The data for the different steps are distinquished by distinctive symbols and the number of the step is indicated on the first point for each line of data. It is seen that the curves for the last four steps agree fairly well. According to the theory of step drawdown tests in confined systems, which we have modified to apply to the unconfined or nonlinear systems, if there were no well loss, these graphs should coincide , except for

FIG. D-21

SEMI-LOG INCREMENTAL DRAWDOWN GRAPH
FOR POWERS WELL



disturbances caused by inhomogenities and variations in permeability or storativity.

The equation for the drawdown during any one step of the test is as follows:

$$\Delta(h_w^2)/h^* = (2.30\,\Delta Q/2\pi Kh^*)\log(2.25\,Kh^*t/S^*r_w^2) + C'Q^2/h_w^2$$

In this case we have divided the square of the head or depth of flow measured in the well by the effective average depth of flow in order to express drawdown in terms of length instead of length squared as before. This is necessary in order to permit addition of the second term in the foregoing equation, which represents the well loss, the first term then representing the formation loss. The well loss is seen to be proportional to some coefficient C' and to the square of the discharge. Also it is assumed to be inversely proportional to the square of the head measured in the well (h_w^2).

Dividing the differential of h^2 by the increment of discharge that produced it and multiplying by the increment of discharge during the first period we get the following relation:

$$\Delta(h_w^2)\ \Delta Q_1/\ \Delta Q = (2.30\ \Delta Q_1/2\pi K)\log(2.25\ Kh^*t/S^*r_w^2)$$

Now this last equation is written assuming C' = 0, or in other words assuming that the well loss is negligable. (In the case of the Powers well we found this to be so, and this would seem to be reasonable in view of the fact that the maximum discharge of the well was only about 293 gpm.)

Failure of the data for the first and second steps of the drawdown curve to conform to the data for the other steps on Figure D-21 is probably attributable to the fact that we have underestimated the discharge during the first period of the test. As indicated in the notes under the foregoing tabulation of data on this test, it was found necessary to interrupt the pumping to change from a 5-inch to a 3-inch orifice because the owner had overestimated the capacity of his well. It is not unlikely that the discharge during the first period of the test was even greater than plotted on the Figure D-20. Actually in obtaining the bottom two curves on Figure D-21 we used a discharge increment of 268 gpm, and the same as for steps 5 and 6. To bring the data into agreement it would be necessary to assume that the discharge increment during the first and second steps was about 28 percent greater than during the fifth and sixth steps, or in orther words about 340 gpm instead of 280 to 293 gpm as recorded using the 5-inch orifice, which proved to be too large.

The change of h^2 over one log-cycle is in this case 18 feet, and the hydraulic conductivity is calculated to be 1,040 ft/day. The drawdown after 100 minutes is about 118 feet and the log of ($S^*r_w^2/h_w$[ft]) about 0.66 - 5. The initial depth to flow was 11.8 feet.

Table D-2 summarizes the data from the step-drawdown test of the Powers well. It gives increments of drawdown and discharge measured 20 minutes after each change in the rate of discharge. The first column gives the number of the step and the second and third columns give the increments of discharge, in gpm and ft^2/day respectively. The fourth and

TABLE D-2

DATA FROM STEP-DRAWDOWN TEST OF POWERS WELL

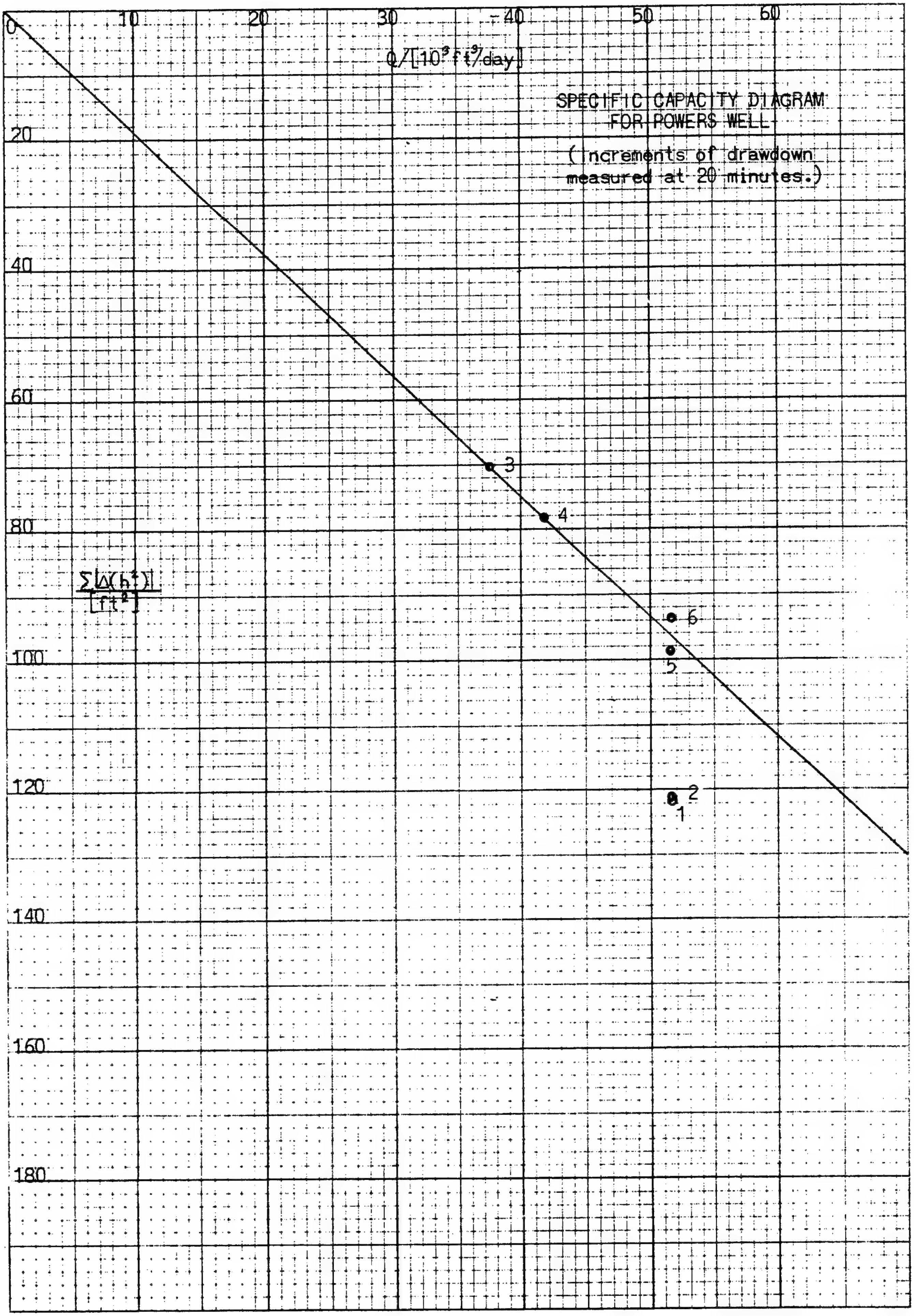
(Increments of drawdown measured at 20 minutes.)

[1]	[2]	[3]	[4]	[5]	[6]	[7]
	Increment of discharge, ΔQ		Discharge $\lvert Q \rvert$		Increment of drawdown	Sum of drawdown increments,
Step	gpm	ft^3/day	gpm	ft^3/day	$\lvert \Delta h^2 \rvert / [ft^2]$	ft^2
1	+268	+51,600	268	51,600	121.2	121.2
2	-268	-51,600	268	51,600	121.6	121.6
3	+195	+37,500	195	37,500	70.5	70.5
4	+ 22	+ 4,300	217	41,800	7.8	78.3
5	+ 51	+ 9,800	268	51,600	20.4	98.7
6	-268	-51,600	268	51,600	93.6	93.6

fifth columns give the actual discharge or sum of increments of discharge, in gpm and ft^3/day respectively. The sixth column of Table D-2 gives the increment of drawdown for each step of the test, expressed as Δh^2 in ft^2. Lastly, the seventh column of the table gives the sum of the drawdown increments in ft^2.

Figure D-22 is a specific-capacity diagram for the Powers well, on which are plotted for each of the six steps of the test the sum of the drawdown increments in square feet against the sum of the discharge increments in thousands of cubic feet per day. It is seen that the points for the last four steps of the test fall fairly close to a straight line passing through the origin, whereas the points for the first and second steps of the test lie somewhat below that line. The reasons for this are as already stated, namely that the discharge during the first period of the test undoubtedly had been underestimated.

We may conclude from this graph or specific-capacity diagram that, for practical purposes, over the range of discharge involved here, when dealing with wells similar in design to the Powers well, the well loss may be neglected. Departures from linearity of flow in the vicinity of the well and convergence losses as the water descends to the well may be taken care of by high negative values of the log of the effective radius of the well.


0
10
20
30
40
50
60
Q/[10³ ft³/day]
SPECIFIC CAPACITY DIAGRAM
FOR POWERS WELL
(Increments of drawdown
measured at 20 minutes.)
20
40
60
80
100
120
140
160
180
Σ[Δ(h²)]
[ft²]
1
2
3
4
5
6

Interference Test of Powers Well

Theory and procedure. The theory and procedure of interference tests of the kind run on the Powers well is given on page B-3 of Appendix B and on page 6 of the Memorandum of Conference of 9 August 1960. Figure D-23a is a map showing the lay-out for the interference test of the Powers well. Indicated thereon are the locations, well-head elevations and depths for each of the four piezometers, for Well 1072, Well A and the Powers Well itself.

Conduct of test. Well A is connected to the Powers well through a buried siphon. The influence of the pumping from the Powers well on the water level in Well A may be seen on Figure D-28. No attempt was made to correct for this effect, however its neglect is not thought to be of serious consequence.

Before the well was pumped the trend of water levels was established in all the neighboring wells by daily measurements from 5 October through 8 October. The pump was started at 08:23 on 9 October 1960. Pumping continued at the average rate of 264 gpm or about 50,900 ft^3/day until 08:33 on 12 October 1960. Observations of recovery were continued with regularity until about midnight that night. Thus the length of the pumping period was about 4,300 minutes, and of the recovery period about 950 minutes. Further infrequent checks on the recovery were made on the succeeding days, however.

The drawdown and recovery observed in the Powers well and in surrounding wells during the test from October 9 to 13 are plotted on Figures D-24 through D-30, as follows:

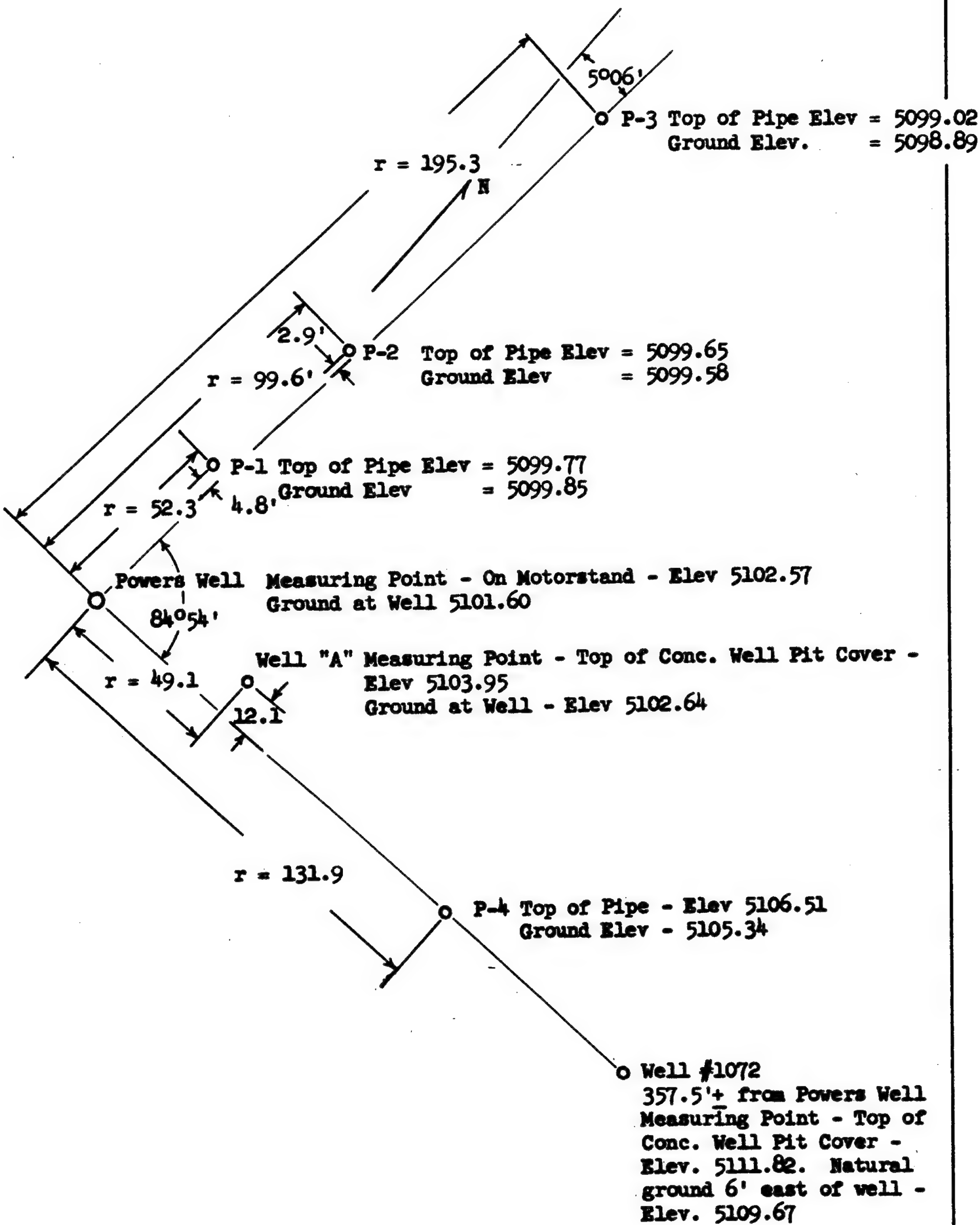
D-24	Drawdown and Recovery Graph of Powers Well	
D-25	Graph Showing Interference of Powers Well on	P1
D-26		P2
D-27		P3
D-28		Well A
D-29		P4
D-30		Well 1072

Analysis of data. Shown on each of these graphs but two, as a dashed inclined line near the top of the graph, is the extrapolated initial trend of the water level. The drawdown is obtained by taking differences at any time between the values of h^2 from this line and the values of h^2 read from the curve drawn through the average of the plotted points. Similarly the recovery is taken as the difference at any time between the h^2 read from the recovery curve traced through the plotted points and the extension of the drawdown curve at the same time. These differences in h^2, in square ft, are plotted on the semi-log graphs in Figures D-31 through D-35. These graphs are as follows:

D-31	Semi-log Drawdown Graph of Powers Well	
D-32	Drawdown	P1, P2 and P3
D-33	Recovery	P1, P2 and P3

POWERS WELL:

MAP OF PUMPING TEST LAYOUT FOR POWERS WELL



GEOLOGIC CROSS-SECTION AT POWERS WELL

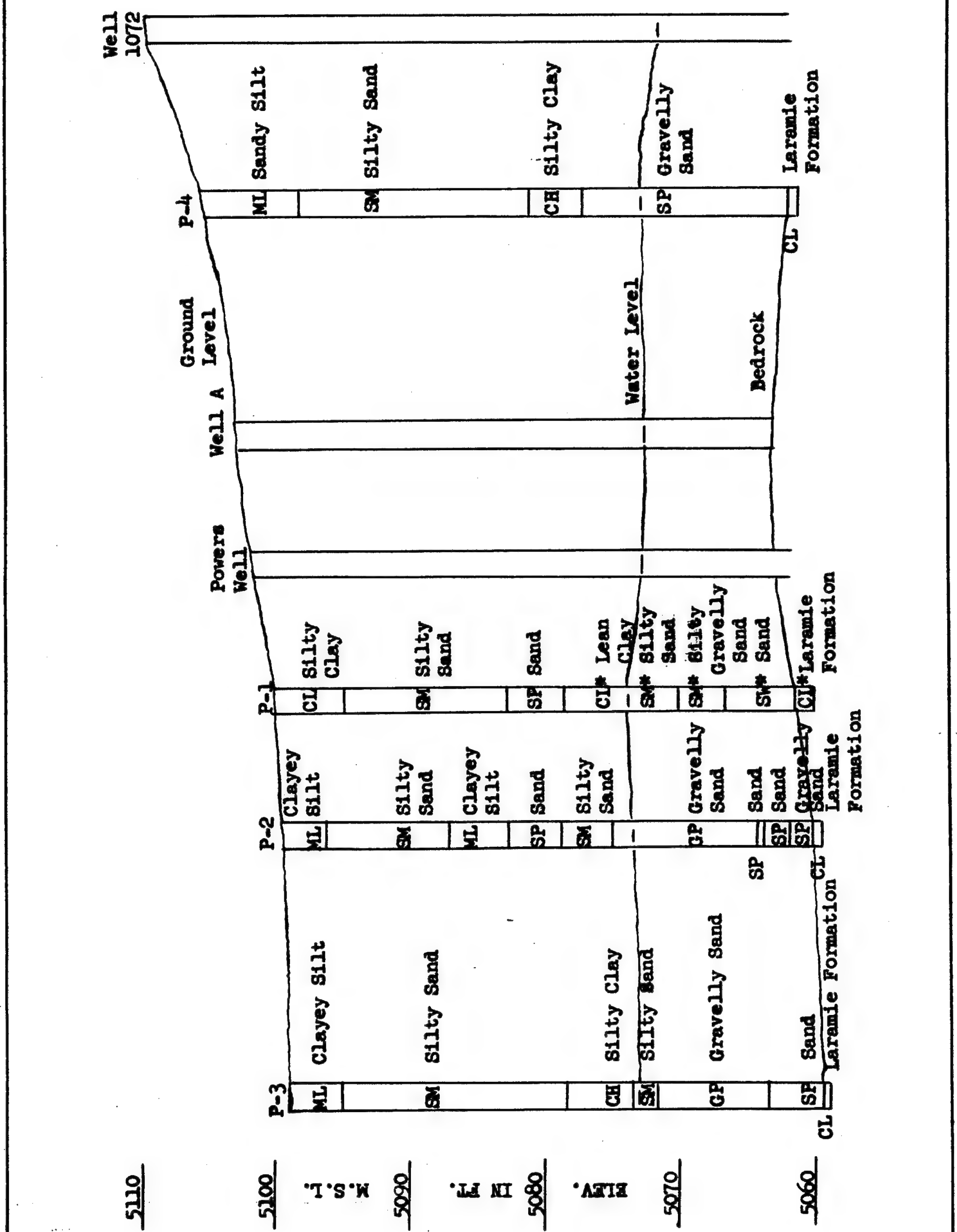
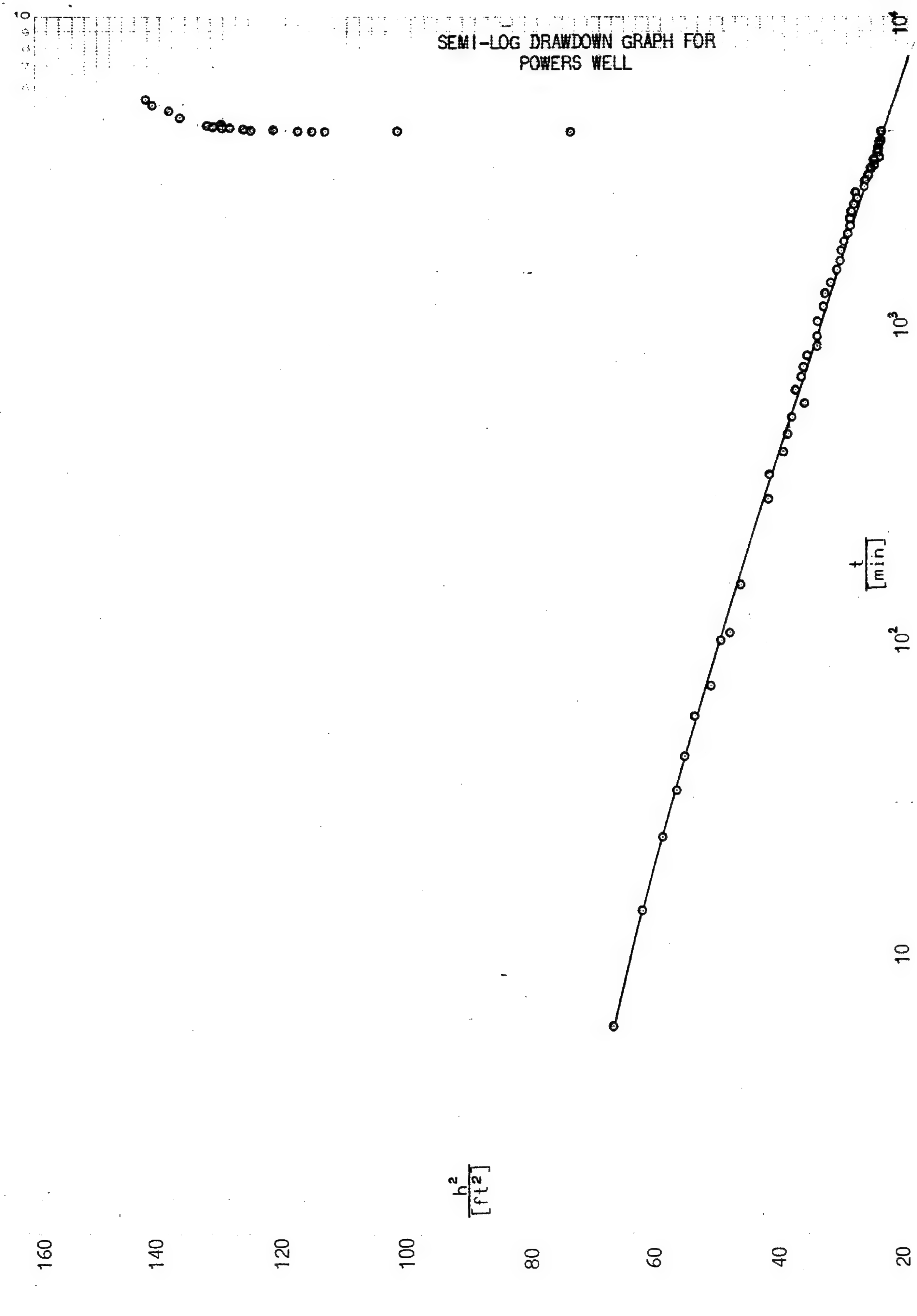
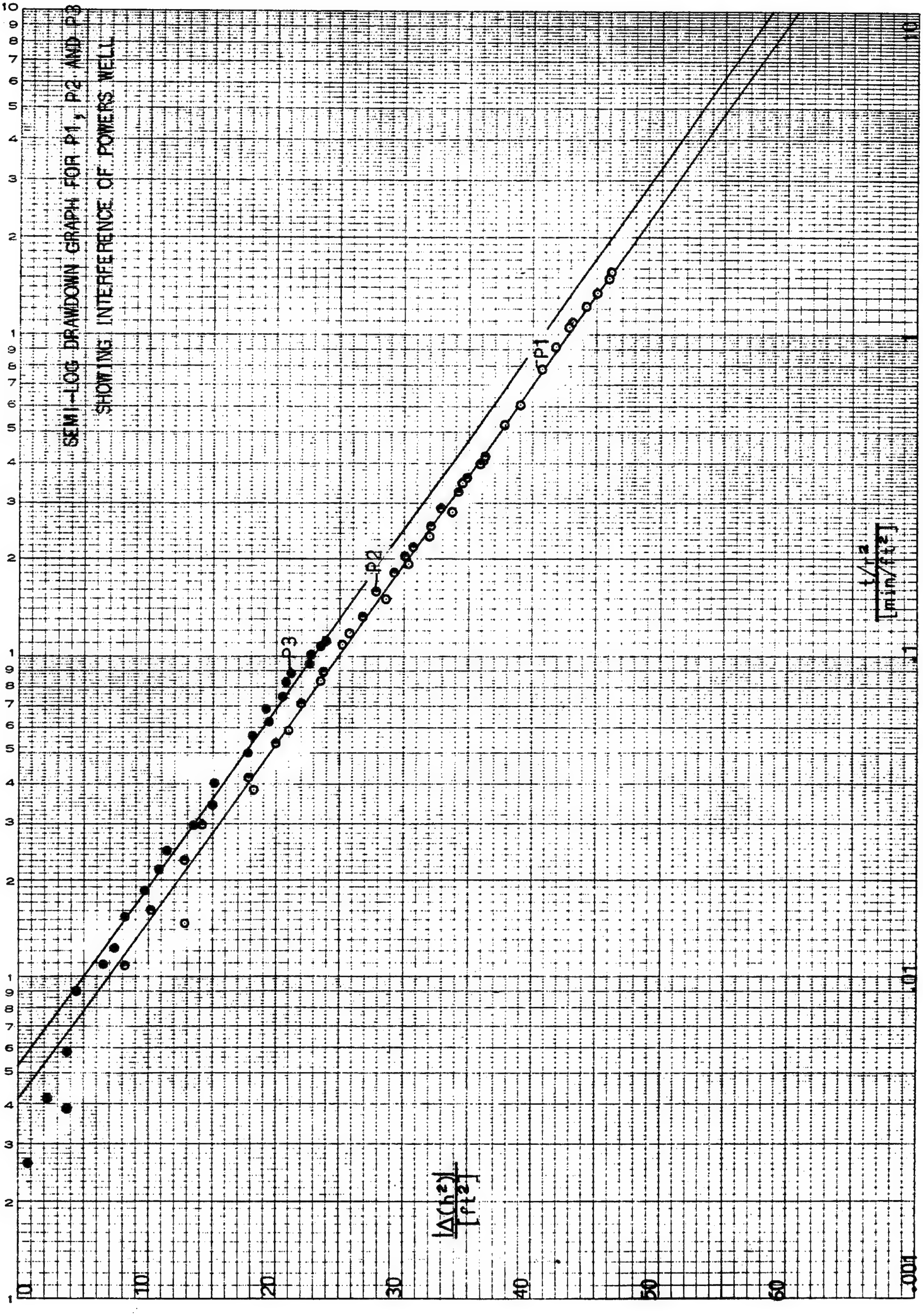


FIG. D-31



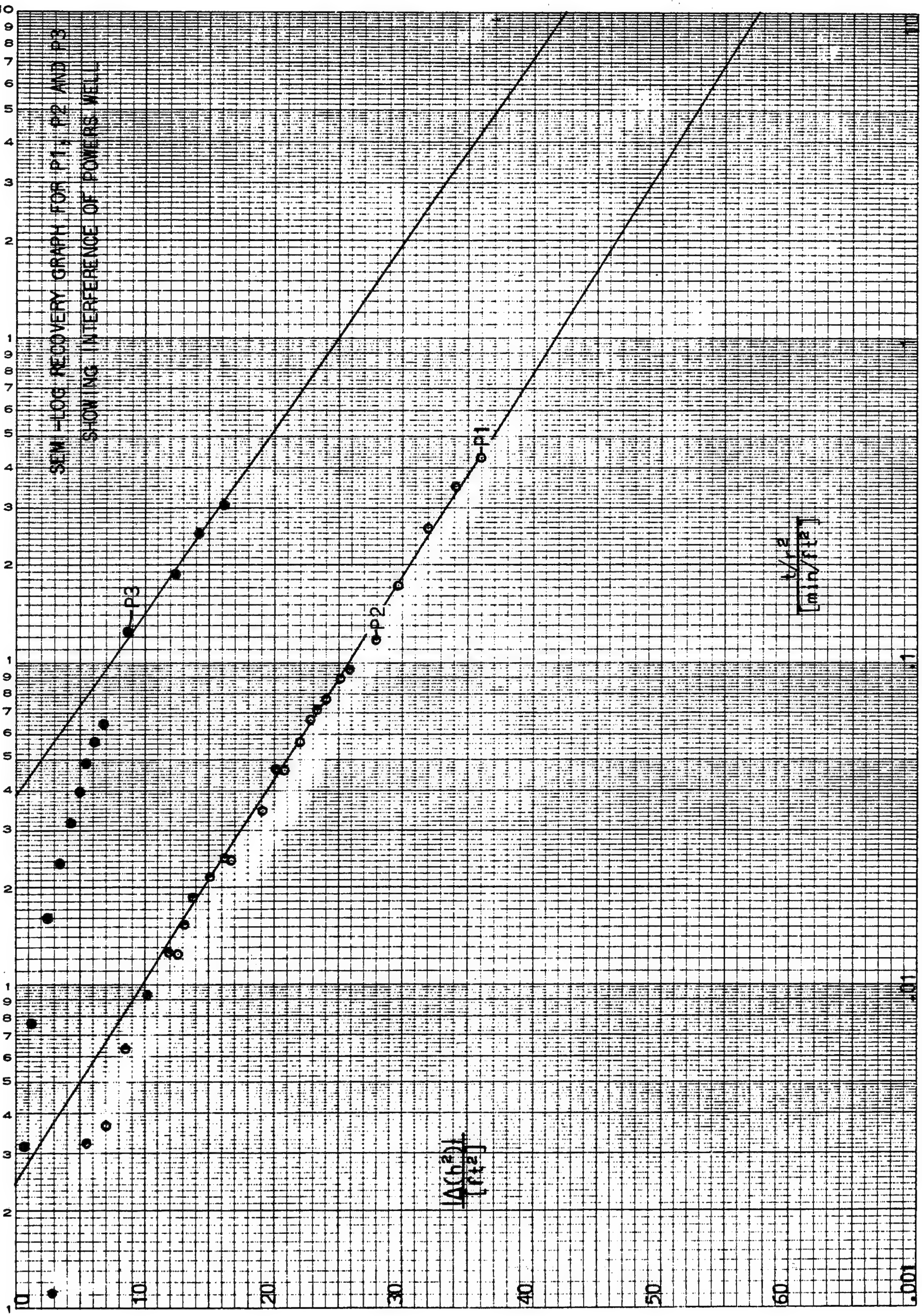
SEMI-LOG DRAWDOWN GRAPH FOR
POWERS WELL
h² [ft²]
160
140
120
100
80
60
40
20
t [min]
10
10²
10³
10⁴

FIG. D-32



SEMI-LOG DRAWDOWN GRAPH FOR P1, P2 AND P3
SHOWING INTERFERENCE OF POWERS WELL
P1
P2
P3
$\left[\frac{\Delta(h^2)}{ft^2}\right]$
$\left[\frac{t/r^2}{min/ft^2}\right]$

FIG. D-33


SEMI-LOG RECOVERY GRAPH FOR P1, P2 AND P3
SHOWING INTERFERENCE OF POWERS WELL
P1
P2
P3
t/r^2 [min/ft²]
$\Delta(h^2)$ [ft²]
.001
.01
.1
1.0
0
10
20
30
40
50
60

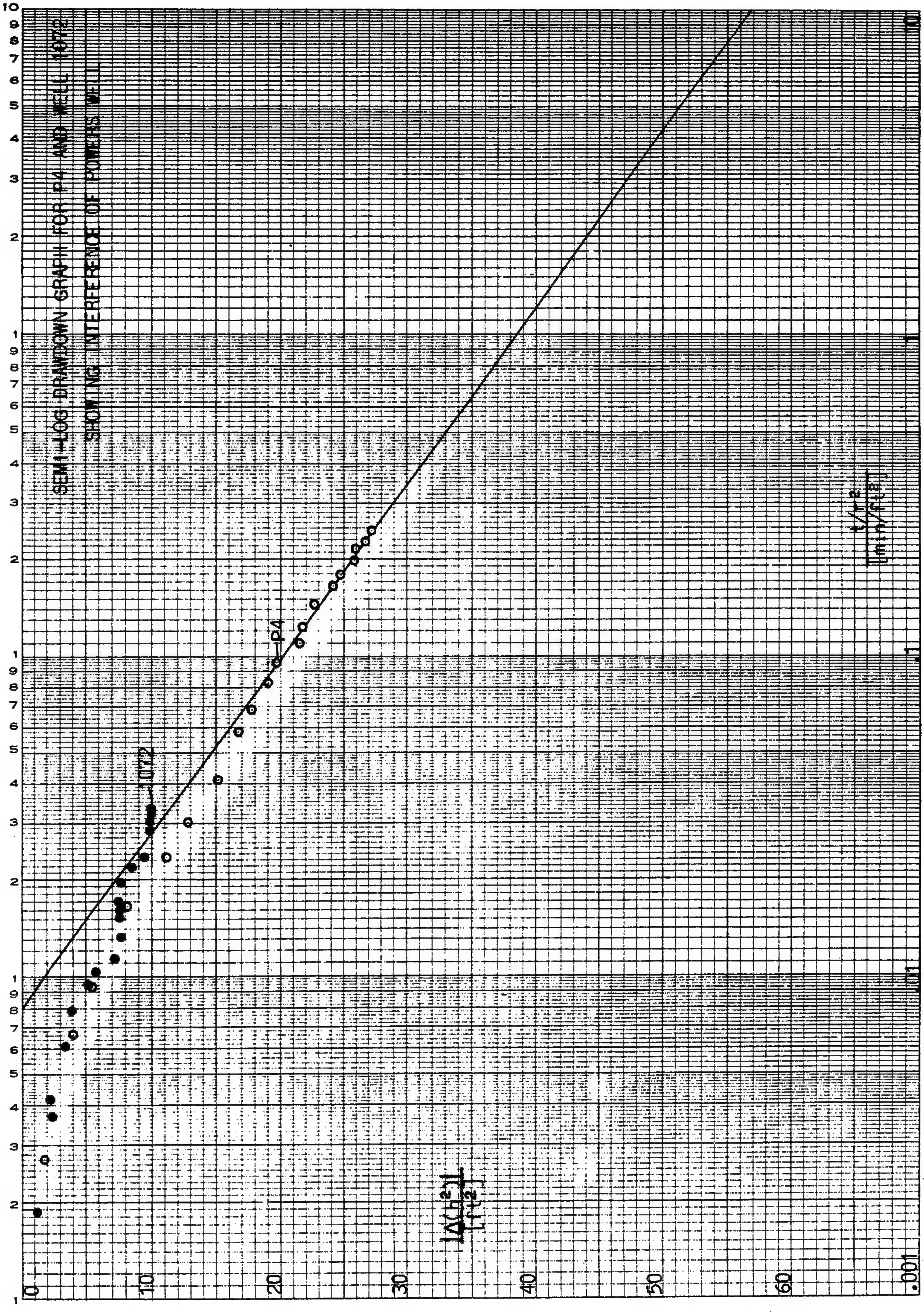
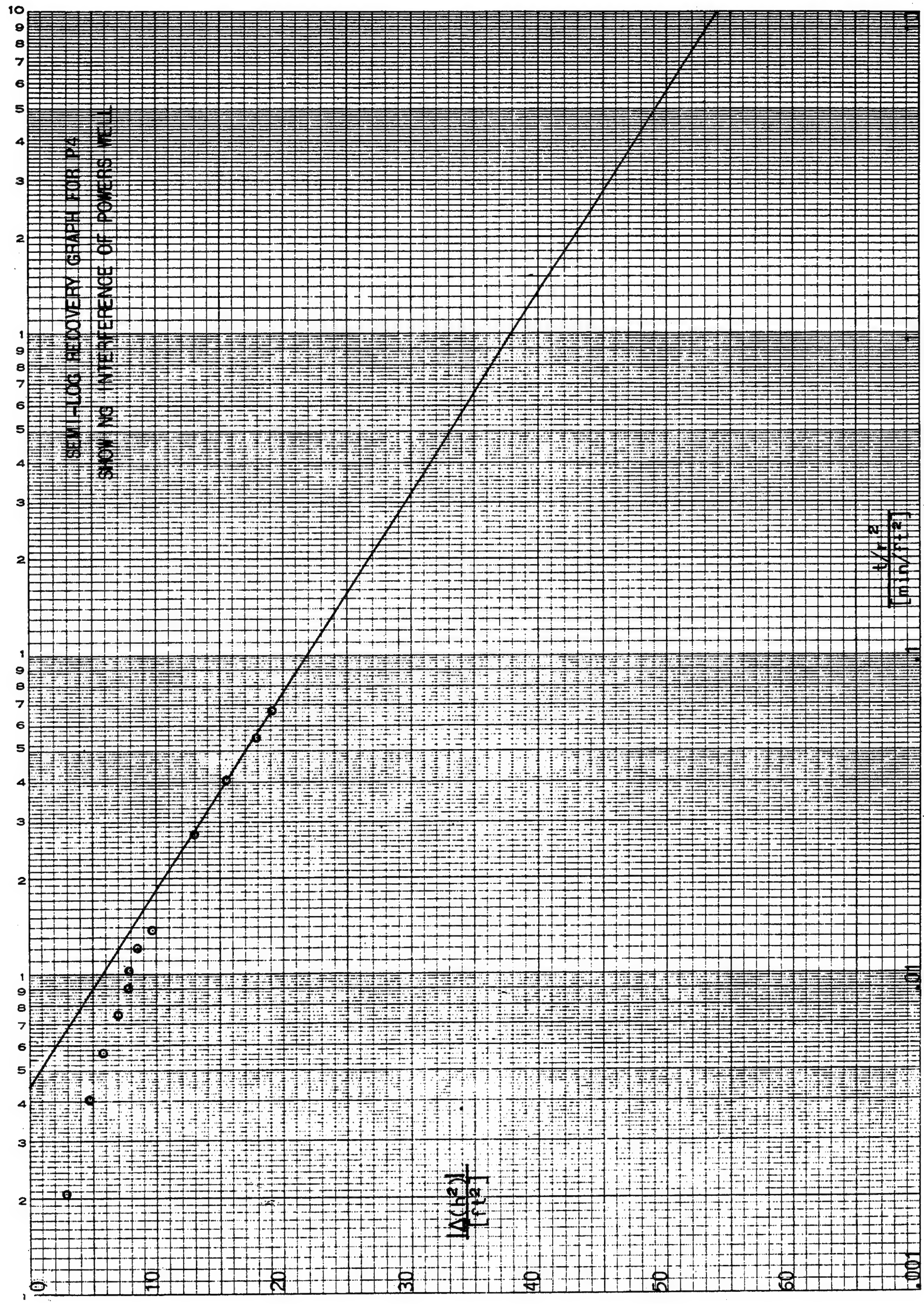

SEMI-LOG DRAWDOWN GRAPH FOR P4 AND WELL 1072
SHOWING INTERFERENCE OF POWERS WELL
t/r²
[min/ft²]
Δ(h²)
[ft²]
P4
1072

FIG. D-34

FIG. D-35


SEMI-LOG RECOVERY GRAPH FOR P4
SHOWING INTERFERENCE OF POWERS WELL
t/r 2
[min/ft2]
Δ(h2)
[ft2]
0
10
20
30
40
50
60
.001
.01
.1
1
10

D-34 Semi-log Drawdown Graph of P4, Well 1072

D-35 Recovery P4

On Figure D-31 the differences in h^2 are plotted as ordinates against the time in minutes on the logarithmic scale. In all the other graphs the logarithmic scale express the ratio (t/r^2) in $[min/ft^2]$. This composite coordinate is used to bring the data from the different wells on a given line into superposition, or as nearly so as the inhomogeneities and variations in storativity permit.

The points on the semi-log drawdown and recovery graphs plot very closely to straight lines after the initial period of expansion of the transient. Straight lines are drawn through the data, in some cases averaging the data for two observation wells. From the slopes of these straight lines it is possible to estimate the hydraulic conductivity, and from their intercept on the zero drawdown line the ratio of effective average depth of flow to effective average storativity may be determined.

The equation for the drawdown expressed as a change of h^2 is as follows:

$$\Delta(h^2) = (2.30\ Q/2\pi K)\log(2.25\ Kh^*t/S^*r^2)$$

The hydraulic conductivity is calculated from the following relation:

$$K = 2.30\ Q\ \Delta(\log t)/2\pi\ \Delta(h^2)$$

At the intercept, $\log = 0$, and $2.25(Kh^*/S^*)\ (t/r^2)_o = 1$.

Solving this for the desired ratio of effective average depth of flow to effective average storativity, one gets

$$h^*/S^* = 1/[2.25\ K(t/r^2)_o]$$

Table D-3 gives a summary of the data from the interference test of the Powers well. The first column gives the name of the observation well, including the Powers well itself as one of these. The second column gives the distance in feet from each observation well or piezometer to the pumping well. No radius is given for the Powers well itself as its effective radius is as yet undetermined.

Column [3] gives the change over one log-cycle of the absolute value of differences in h^2, in ft^2. Drawdown is distinquished from recovery by "d", as against "r".

Values of calculated hydraulic conductivity are given in Column [4] in ft/day. These range from 1,020 to 1,170 and average 1,090.

The intercepts on the zero-drawdown line are given in column [5], and from these values of the ratio (h^*/S^*) given in Column [6] were obtained. Those from the drawdown are listed on the left and those from the recovery on the right. The range here is large, but it must be remembered that it is the logarithm of this ratio that is involved in our calculations or predictions. The average of all six calculated ratios is 116 ft. As the

TABLE D-3

DATA FROM INTERFERENCE TEST OF POWERS WELL

[1]	[2]	[3]	[4]	[5]	[6]
Observation well	Distance $r/[\mathrm{ft}]$	Change over[a] one log-cycle of $\lvert\Delta(h^2)\rvert/[\mathrm{ft}^2]$	Hydraulic conductivity $K/[\mathrm{ft/day}]$	Intercept $\frac{(t/r^2)_0}{[\mathrm{min/ft}^2]}$	$\frac{h^*/S^*}{[\mathrm{ft}]}$[a]
Powers	—	16.5	1,130	—	—
P1, 2	52.3, 99.6	18.0 d	1,040	0.0042	147
		16.0 r	1,170	.0025	219
P3	195.3	18.0 d	1,040	.0053	116
		17.8 r	1,050	.039	16
P4	131.9	18.3 d	1,020	.0081	77
		16.1 r	1,160	.0045	123
Average			1,090		113d 119r

a) The d means drawdown; the r means recovery.

effective average depth to flow cannot exceed the initial depth to flow, namely 11.8 ft, we will certainly overestimate the average storativity for the period of pumping if we use 11.8 ft for h^*. Doing this, we get the following result $S^* = 11.8 \text{ ft}/116 \text{ ft} = 0.102$.

Interpretation. It would appear that the average storativity for the 3-day interference test is about 10 percent. The storativity remained essentially constant throughout this period, indicating by its constancy and low value that there may be fine-grain sediments in the upper part of the aquifer overlying the more permeable gravel that cause capillary retention. It is not known how extensive this condition is. If, as in this case, up to as much as 20 percent of the total volume may be occupied by capillary water, there may be contaminants or toxicants held above the water table that have not been inventoried by our volumetric calculations.

Interference Test of Aden Well

Theory and procedure. Same as before.

Conduct of the test. The Aden well was pumped continuously for about 4,300 minutes, beginning 16 November 1960 and ending 19 November 1960. The recovery was measured at frequent intervals for another 1800 minutes and daily thereafter for several days.

Three piezometers were drilled on each of two lines, one on an azimuth of 44° 29' and the other on an azimuth of 135° 10'. Their respective distances from the Aden well are as follows:

NE line			SE line		
P1	P2	P3	P4	P5	P6
50.2	100.2	199.3	51.3	98.9	200.6 ft

Water level readings made in the Aden well and in the six piezometers, converted to read in terms of the square of the depth of flow (h^2), in ft^2, are plotted on Figures D-36a through D-38b.

Except for the 35-minutes failure in the pump operation at about 2150 minutes after the start of pumping, the pumping was continous.

Analysis of data. The drawdown curves were extrapolated through the period of recovery and the recovery measured up from that extrapolated drawdown. Values of these differences in h^2, in square feet, are plotted on the semi-log graphs in Figures D-39 through D-44. These graphs are as follows:

D-39	Semi-log Drawdown Graph for Aden Well	
D-40	Recovery	Aden Well
D-41	Drawdown	P1, P2 and P3
D-42	Recovery	P1, P2 and P3
D-43	Drawdown	P4, P5 and P6
D-44	Recovery	P4, P5 and P6

Straight lines are drawn through the points plotted on the semi-log drawdown and recovery graphs. From the slopes of these straight lines the hydraulic conductivity is calculated, and from their intercepts on the zero-drawdown line, the storativity is estimated.

Table D-4 summarizes the data from this interference test of the Aden well. The first column gives the name of the observation well and the second its distance from the pumping well, in feet. The third column gives the change over one log-cycle of h^2, in ft^2.

The average change of h^2 over one log-cycle is 11.1 ft^2 for the five pairs of example of drawdown and recovery. Excluding those for the Aden well itself, the average is 9.9 ft^2. The average intercept, weighted according to the number of observation wells contributing to each number, is 0.0026.

TABLE D-4

DATA FROM INTERFERENCE TEST OF ADEN WELL

[1] Observation well	[2] Distance r/[ft]	[3] Change over one log-cycle of $\lvert\Delta(h^2)\rvert$/[ft^2] [a]	[4] Intercept $(t/r^2)_o$ [min/ft^2]
Aden	—	15.0 d*	—*
		16.8 r*	—*
P1, 2, 3	50.2, 100.2, 199.3	13.0 d	.0047
		8.4 r	.0016
P4	51.3	11.0 d	.0035
		9.7 r	.0024
P5	98.9	8.5 d	.0017
		10.0 r	.0018
P6	200.6	8.5 d	.0017
		10.0 r	.0009
Average		11.1	
---excluding starred numbers		9.9	.0026

a) The d means drawdown; the r means recovery.

Average initial depth of flow, 11.2 ft

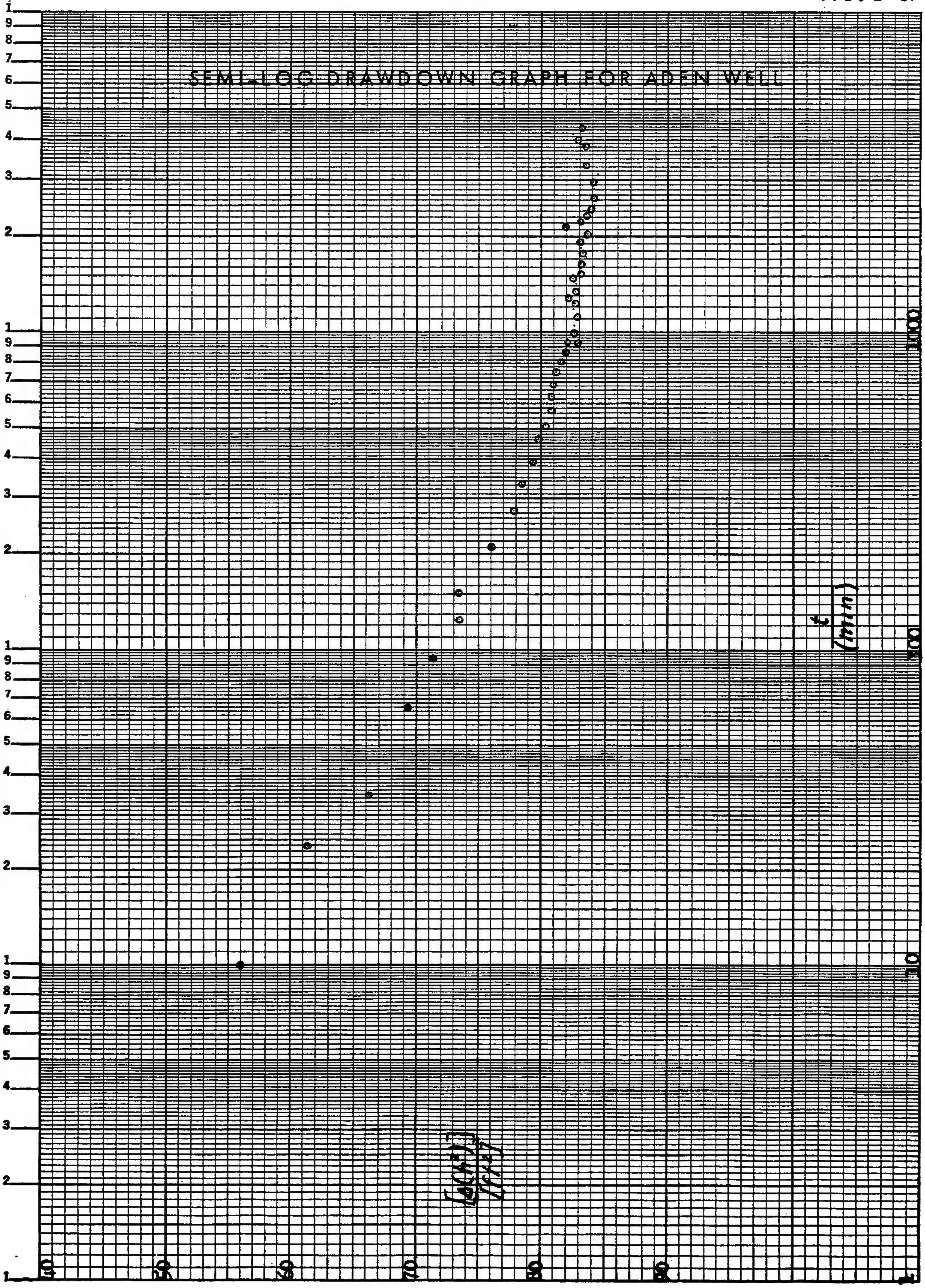
Average hydraulic conductivity

$$K = 2.30\ Q/2\pi\Delta(h^2) = 1{,}230 \text{ ft/day}$$

Average initial apparent storativity

$$S^* = 2.25\ Kh^*(t/r^2)_o = 2.25 \times 1{,}230 \text{ ft/day} \times 11.2 \text{ ft} \times .0026 \text{ min/ft}^2 = .056$$

FIG. D-39


SEMI-LOG DRAWDOWN GRAPH FOR ADEN WELL
t (min)
1000
100
10
1
40
50
60
70
80
90

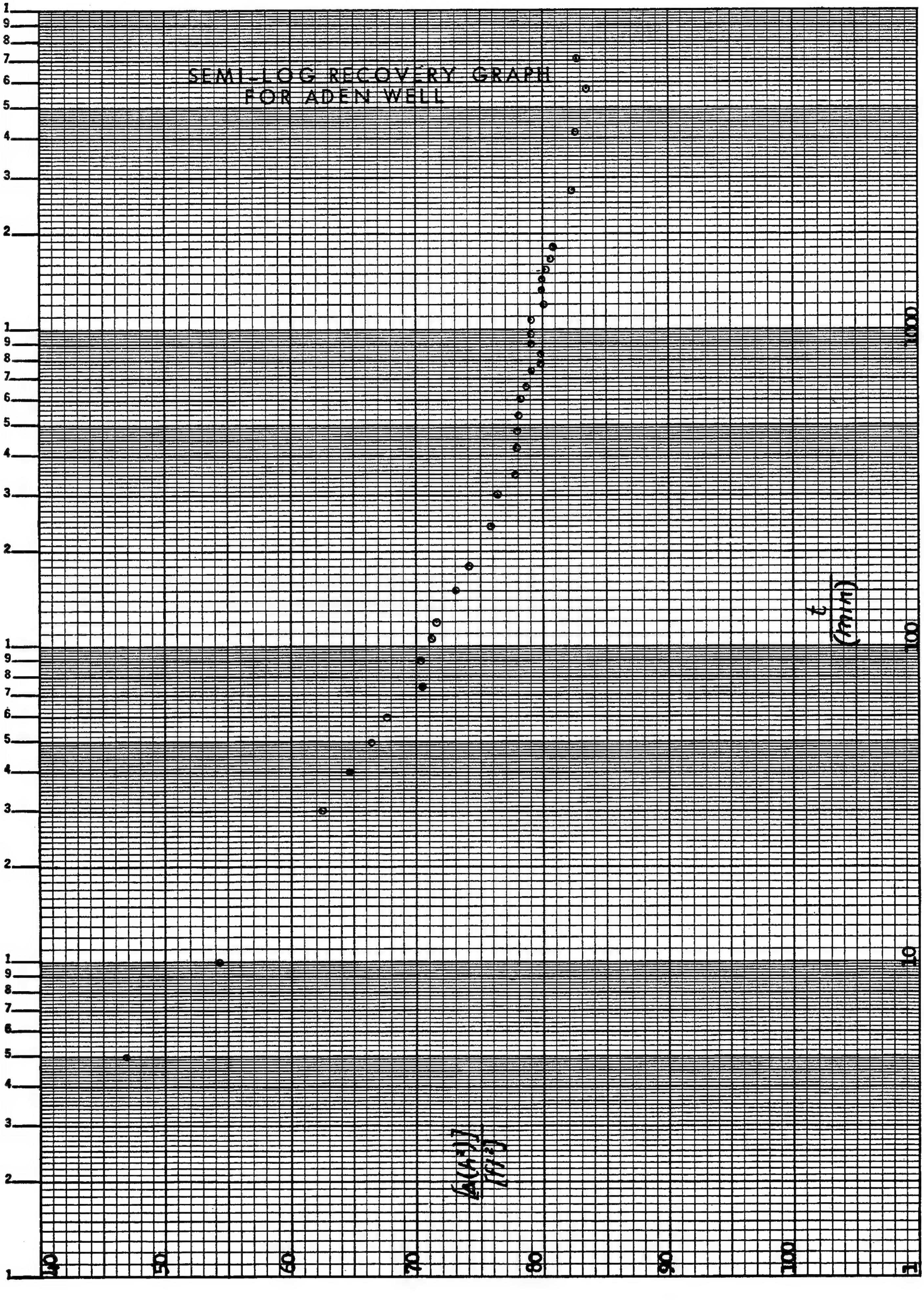

SEMI-LOG RECOVERY GRAPH
FOR ADEN WELL
t (min)
1000
100
10
1
40
50
60
70
80
90
100
110

FIG. D-41

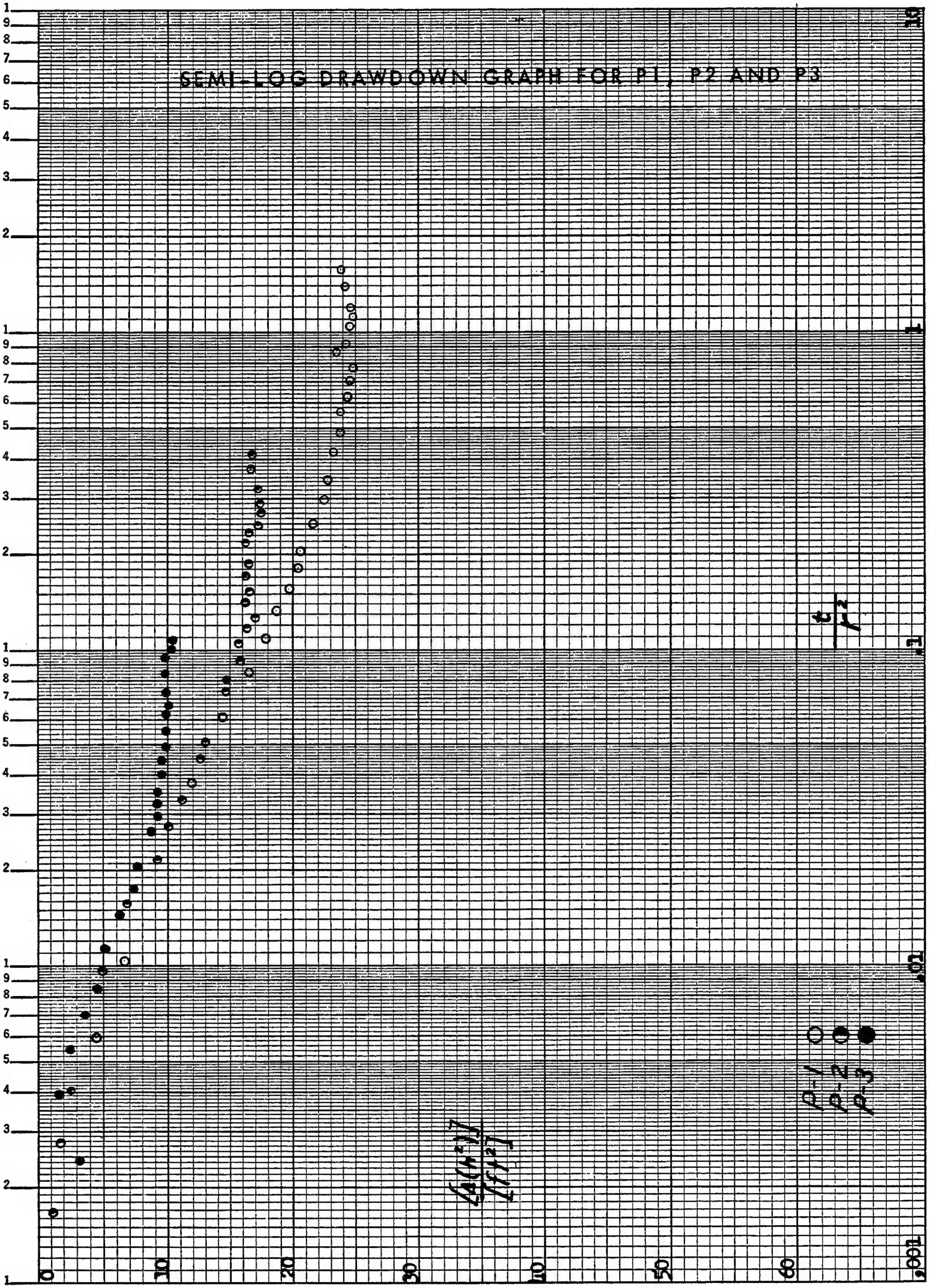


FIG. D-42

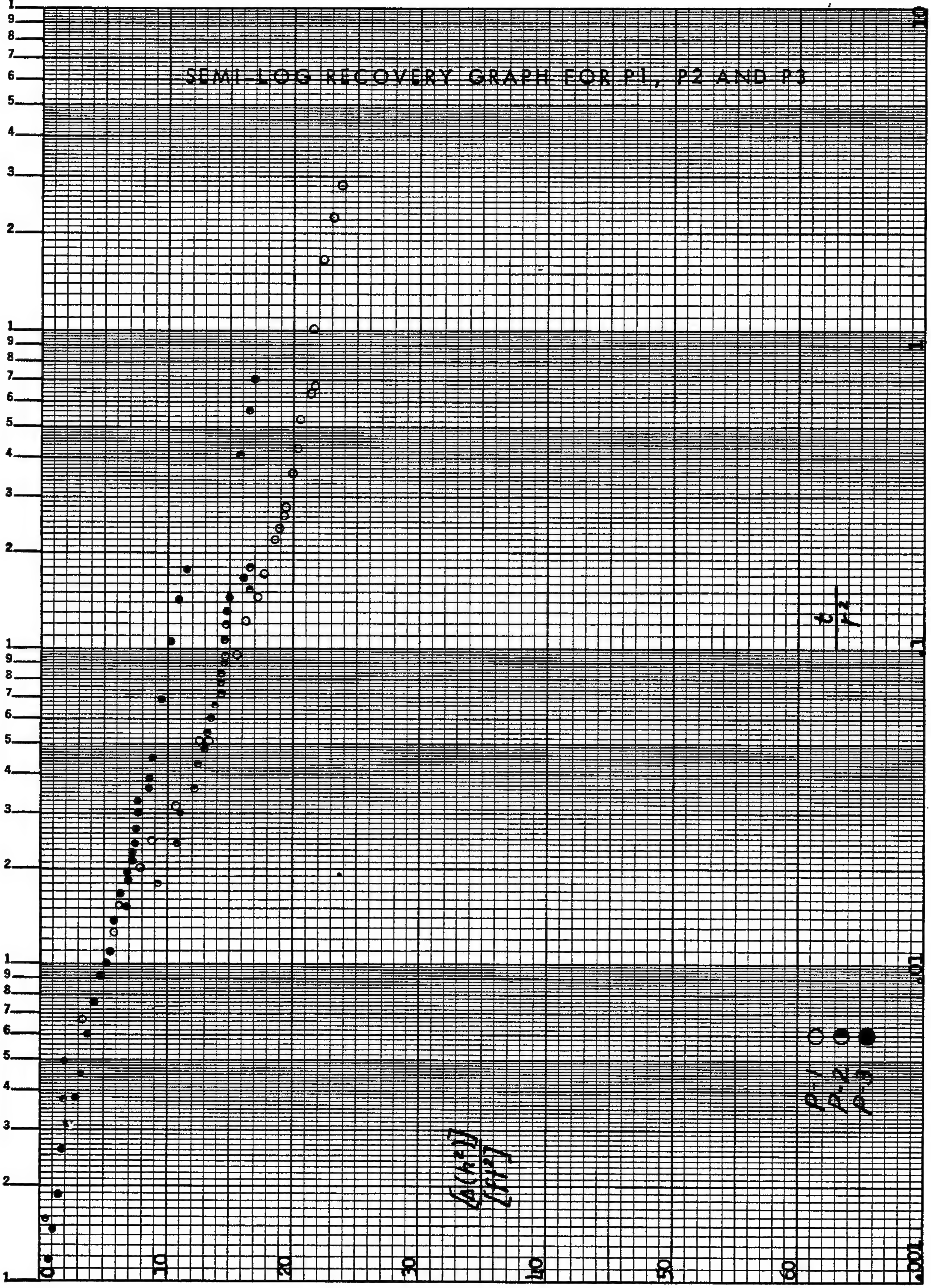

SEMI-LOG RECOVERY GRAPH FOR P1, P2 AND P3
P-1
P-2
P-3
t/r²
Δ(h²)/(r²)
10
20
30
40
50
60
.001
.01
.1
1
10

FIG. D-43

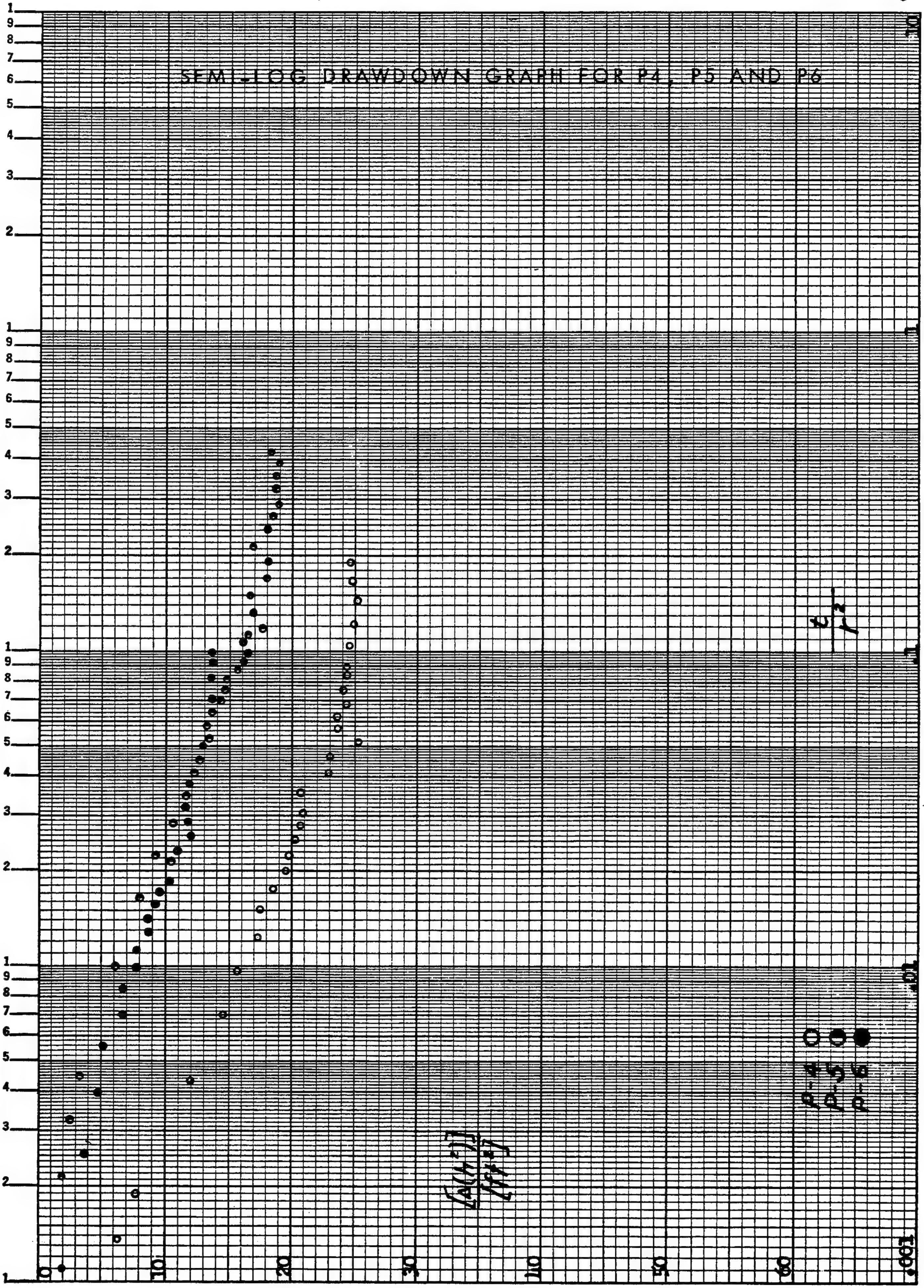
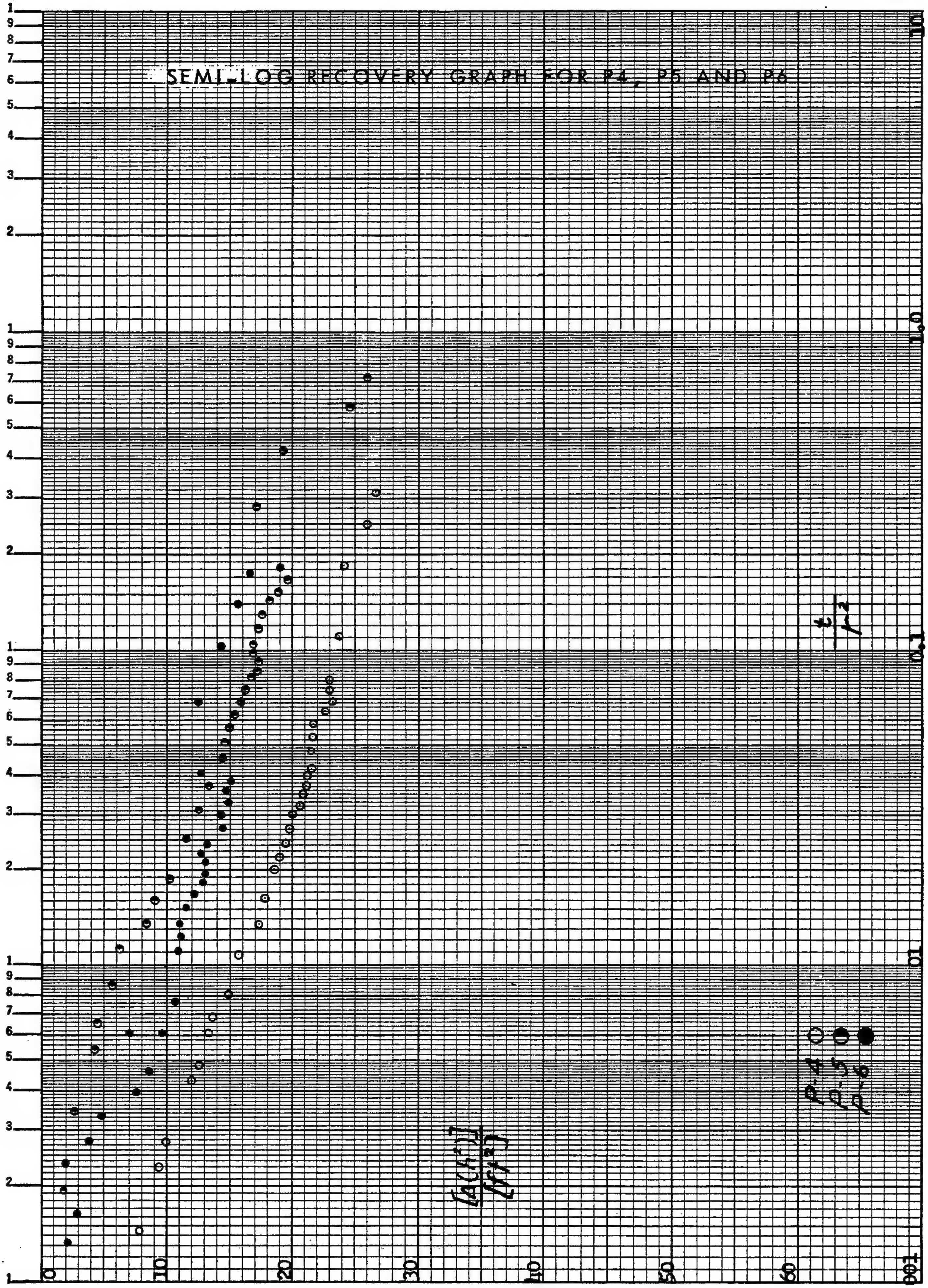


FIG. D-44


SEMI-LOG RECOVERY GRAPH FOR P4, P5 AND P6
$\frac{t}{r^2}$
$\frac{[\Delta(h^2)]}{[r^2]}$
P-4
P-5
P-6
0
10
20
30
40
50
60
.001
.01
0.1
1.0
10

The average discharge was 173 gpm or 33,300 ft^3/day. Thus the hydraulic conductivity is calculated to be about 1,230 ft/day.

Interpretation. Knowing the hydraulic conductivity, it is possible to substitute back in the formula for the drawdown and calculate the effective average storativity (S*). The results of such calculations are depicted on Figure D-45, which is a graph of the variation of apparent storativity during the drawdown, and on Figure D-46, which is a similar graph for the apparent storativity during recovery. Two things are apparent: first, the storativity increases in time and continues to increase throughout the 3-day drawdown period and 5-day recovery period; second, the apparent storativity generally is greater on recovery than on drawdown. Moreover, the apparent storativity is greater along the northeast line (P1, 2 and 3) than it is along the southeast line (P4, 5 and 6). This suggests the possibility of anisotropy as regards permeability, for we arrived at these storativities by using a uniform hydraulic conductivity calculated from the average change over one log-cycle of drawdown in time (see Column [3], Table D-4). The slope of the semi-log time-drawdown graph for P1, 2 and 3 is 13.0 ft^2, while that on the semi-log time recovery graph is only 8.4 ft^2, the average being 10.7 ft^2. On the other hand, the average slope of the semi-log time-drawdown graphs for P4, 5 and 6 is 9.3 ft^2, and of the semi-log time-recovery graphs 9.9 ft^2. As the difference in apparent storativity on the two lines is greater than the difference in hydraulic conductivity, it is evident that that difference in apparent storativity is not due to our assuming a uniform hydraulic conductivity for both lines. Rather it would appear that the degree of confinement and freedom of drainage is different in the general neighborhood of P1, 2 and 3 from what it is in the vicinity of P4, 5 and 6.

Table D-5 summarizes the variation of apparent storativity shown graphically on Figures D-45 and D-46. After 4,000 minutes, or less than three days, the apparent storativity on the northeast line had reached about 17 percent and was still rising, whereas that on the southeast line had reached about 10.7 percent. At the end of about 7,000 minutes, or less than three days of recovery, the apparent storativity on the northeast line had exceeded 26 percent and was still rising sharply, whereas that on the southeast line had reached something more than 8 percent and was rising less rapidly.

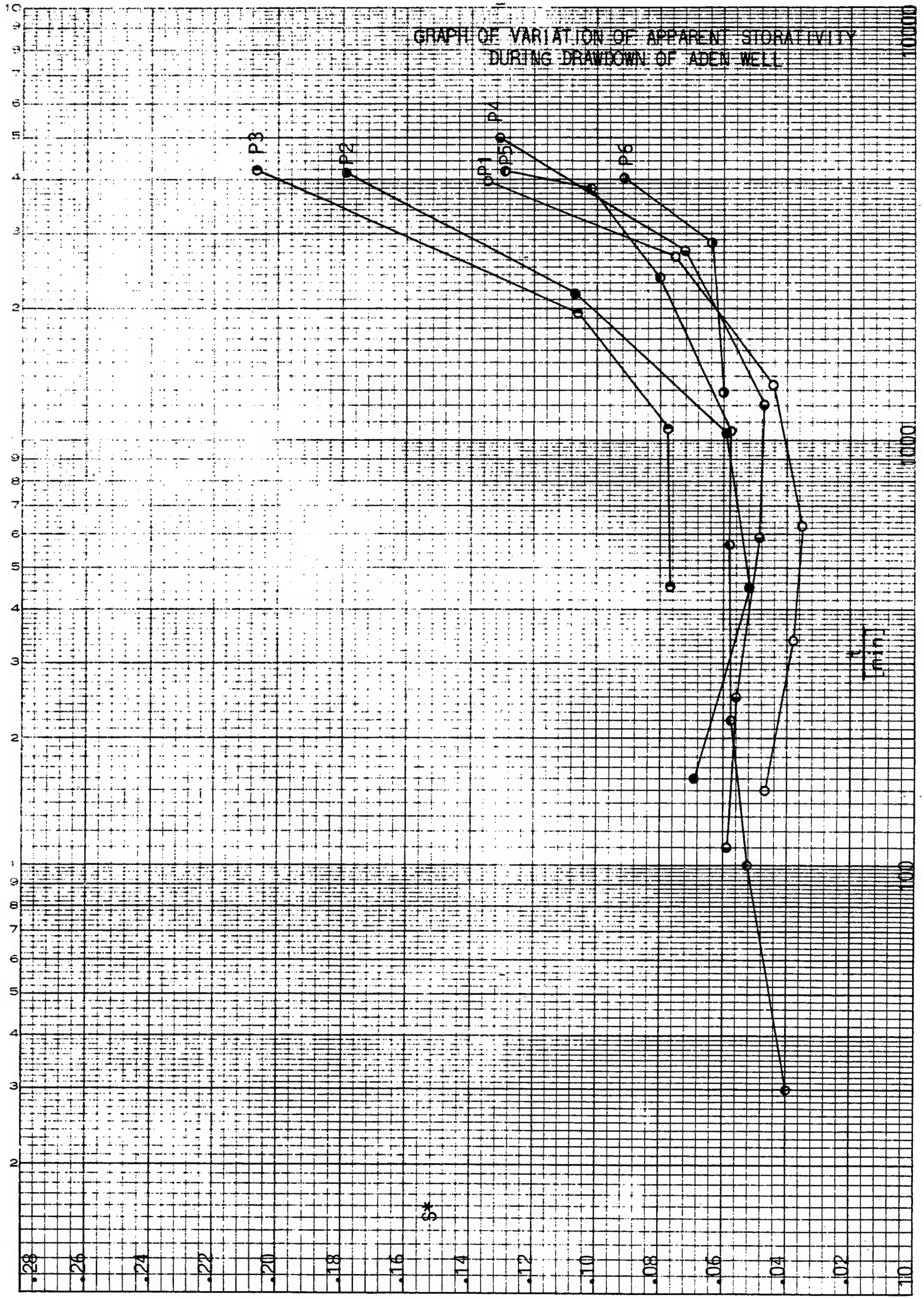
It is now evident that in order to be able to estimate ultimate storativity or "specific yield" it would be necessary to measure interference over these distances for a few weeks. Such tests are not now feasible but may become so during the decontamination operation in the future. Pending that we may hazard a guess that on the northeast line the ultimate apparent storativity would reach 35 percent and perhaps half that on the southeast line.

TABLE D-5

VARIATION OF APPARENT STORATIVITY
IN TIME, NEAR ADEN WELL

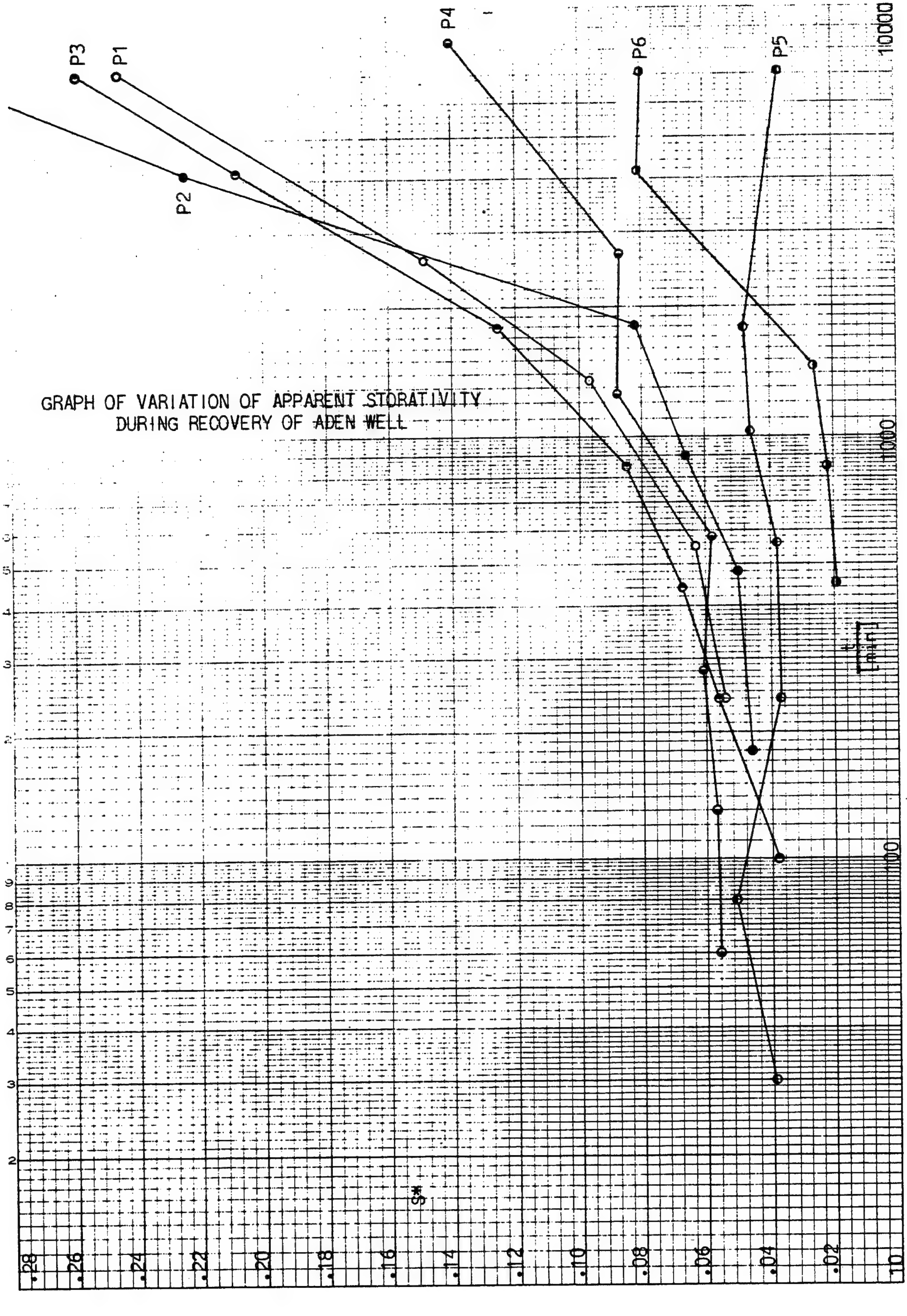
[1]	[2]	[3]	[4]	[5]
	Storativity during			
	Drawdown		Recovery	
Time t [min]	NE line (P1, 2, 3)	SE line (P4, 5, 6)	NE line (P1, 2, 3)	SE line (P4, 5, 6)
100	0.053	----	0.038	0.052
1,000	.059	.053	.082	.049
2,000	.092	.067	.121	.058
4,000	.170	.107	.206	.076
7,000	----	----	.266	.083
Estimated ultimate		.18	.35	

FIG. D-45



GRAPH OF VARIATION OF APPARENT STORATIVITY
DURING DRAWDOWN OF ADEN WELL
P1
P2
P3
P4
P5
P6
t min.
S*
.28
.26
.24
.22
.20
.18
.16
.14
.12
.10
.08
.06
.04
.02
10
100
1000
10000

FIG. D-46



GRAPH OF VARIATION OF APPARENT STORATIVITY
DURING RECOVERY OF ADEN WELL

ADEN WELL:

MAP OF PUMPING TEST LAYOUT FOR ADEN WELL

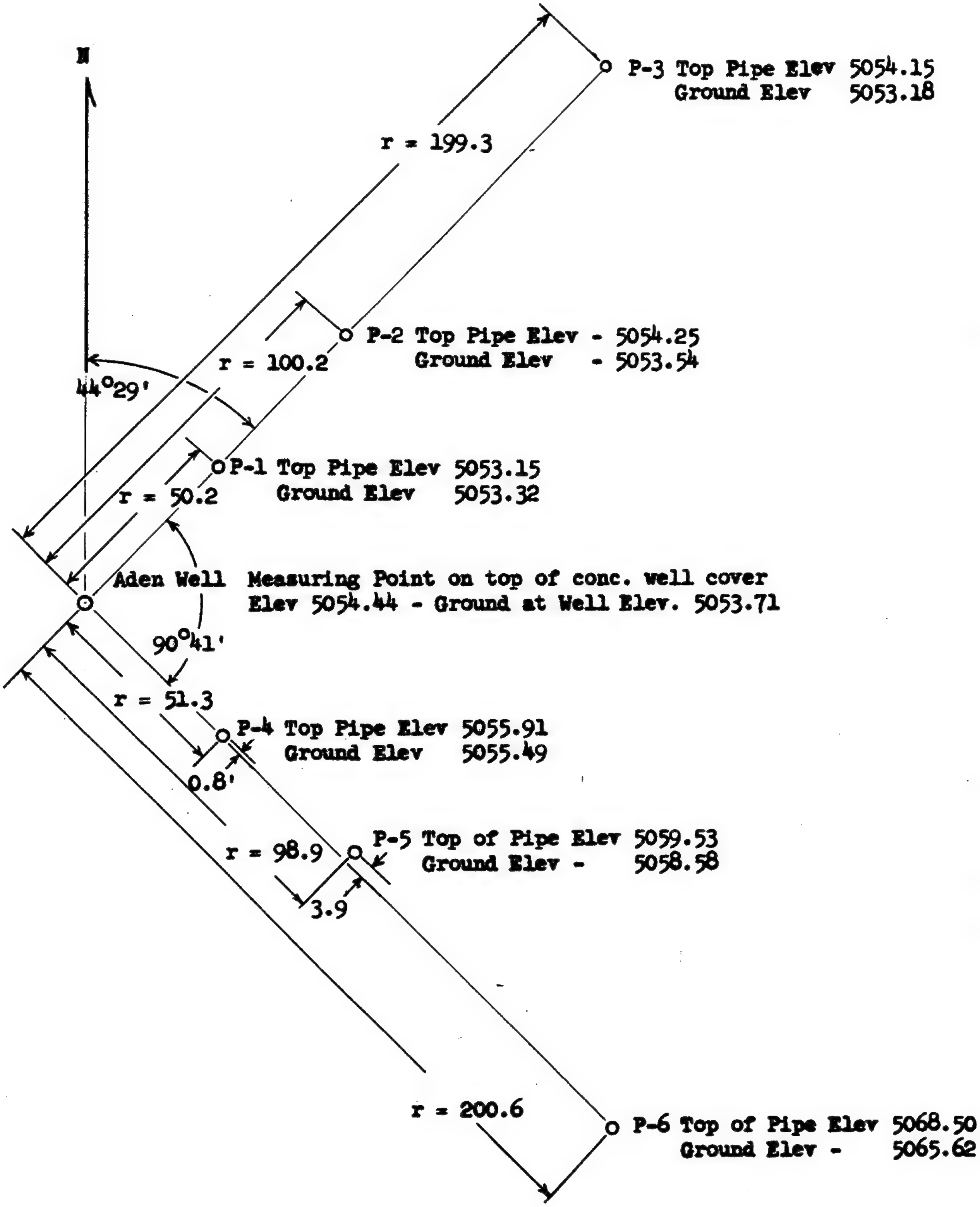
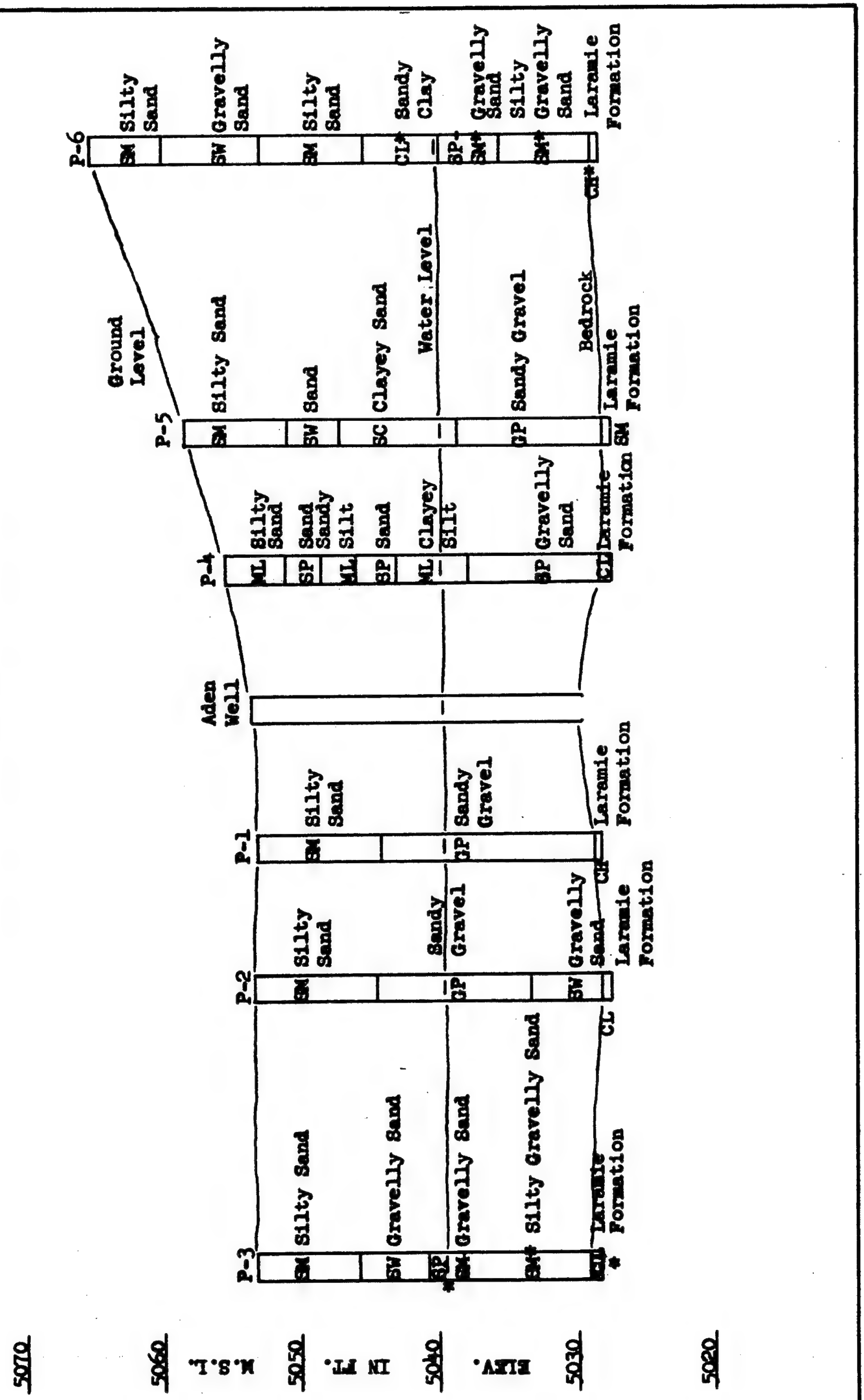


FIG. D-47b



GEOLOGIC CROSS-SECTION AT ADEN WELL

Interference Test of Marty Well

Theory and procedure. Same as before

Conduct of test. The Marty Well was pumped continuously for about 4,400 minutes. The recovery was measured at frequent intervals for another 3,300 minutes and daily thereafter for several days.

Three piezometers were drilled on each of two lines, one bearing approximately southeast and the other one approximately southwest. (See map on Figure D-48a.) The distances from the Marty Well to the piezometers are given in column 2 of Table D-6.

Water-level readings made in the Marty Well and six piezometers, converted to read in terms of the square of the depth of flow (h^2), in ft^2, are plotted on Figures D-49a through D-55b.

Analysis of data. The drawdown curves were extrapolated through the period of recovery and the recovery measured up from that extrapolated drawdown. Values of these differences in h^2, in square feet, are plotted on the semi-log graphs in Figures D-56 through D-61. These graphs are as follows:

D-56	Semi-log	Drawdown	Graph for	Marty Well
D-57		Recovery		Marty Well
D-58		Drawdown		P3, P2 and P1
D-59		Recovery		P3, P2 and P1
D-60		Drawdown		P4, P5 and P6
D-61		Recovery		P4, P5 and P6

Straight lines are drawn through the points plotted on the semi-log drawdown and recovery graphs. From the slopes of these straight lines the hydraulic conductivity is calculated, and from their intercepts on the zero-drawdown line, the storativity is estimated.

Table D-6 summarizes the data from this interference test of the Marty Well. The first column gives the name of the observation well and the second its distance from the pumping well, in feet. The third column gives the slope of the line on the semi-log graph, in feet squared.

The average discharge during the test was about 638 gpm or 123,000 ft^3/day. From the known discharge and from the slopes given in column 3, values of hydraulic conductivity in column 4 were obtained. These range from 1,050 to 1,800 ft/day and average about 1,330 ft/day.

The fifth column of Table D-6 gives the intercepts on the zero-drawdown line in min/ft^2. From these intercepts values of the ratio h^*/S^* in feet, given in column 6, were obtained. The symbol h^* stands for the effective average depth of flow. At any

MARTY WELL:

MAP OF PUMPING TEST LAYOUT FOR MARTY WELL

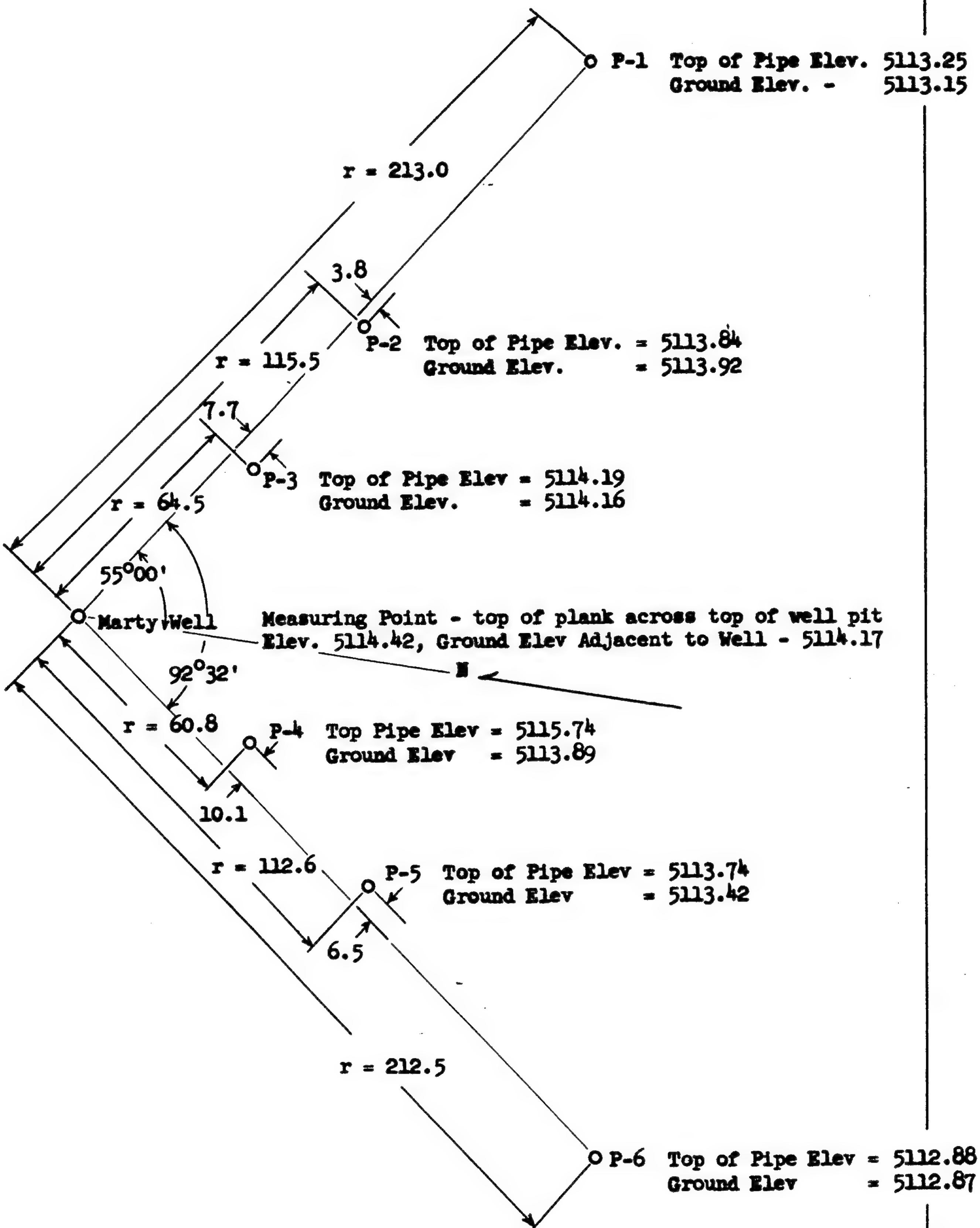
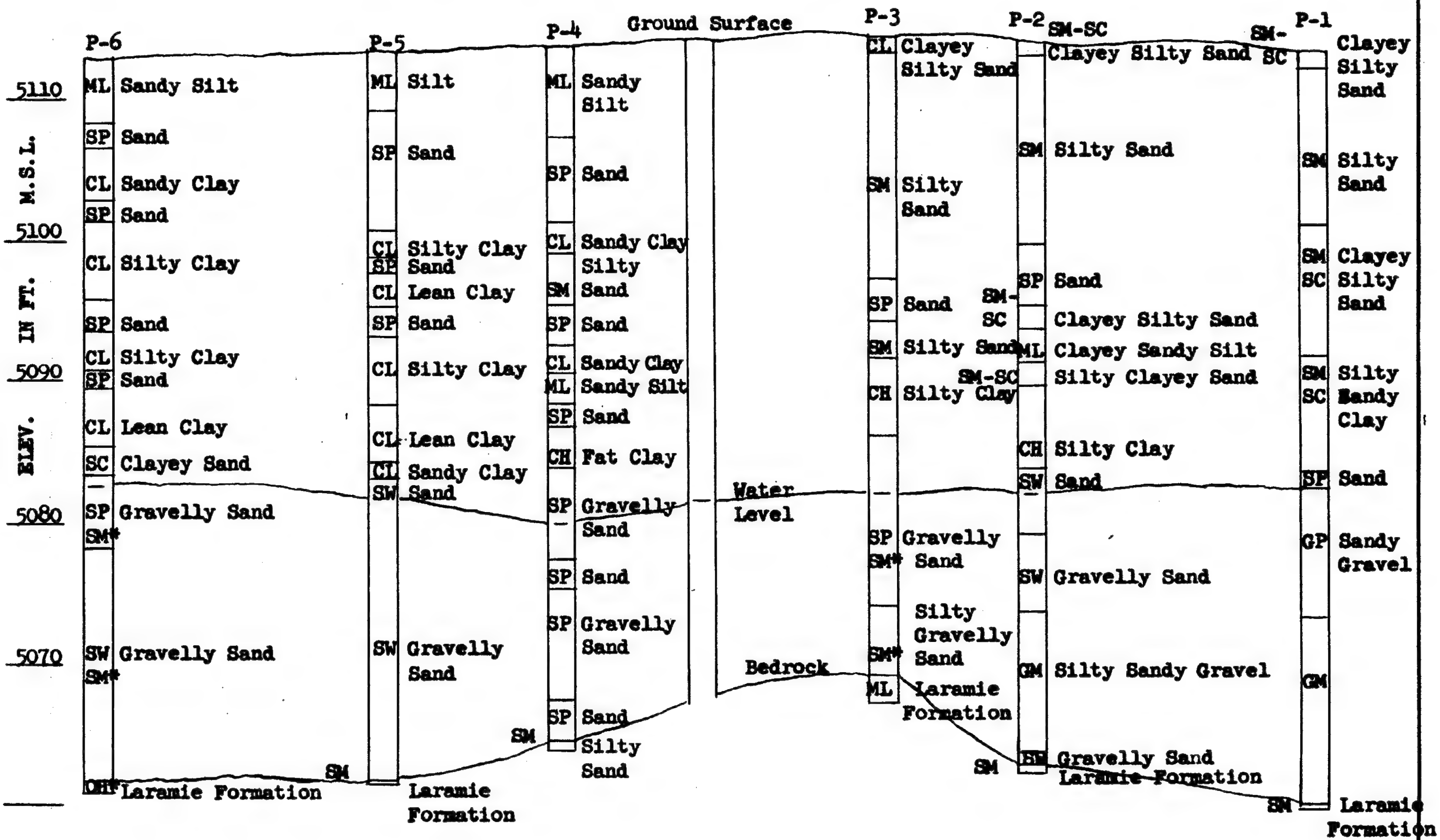


FIG. D-48b

GEOLOGIC CROSS-SECTION AT MARTY WELL



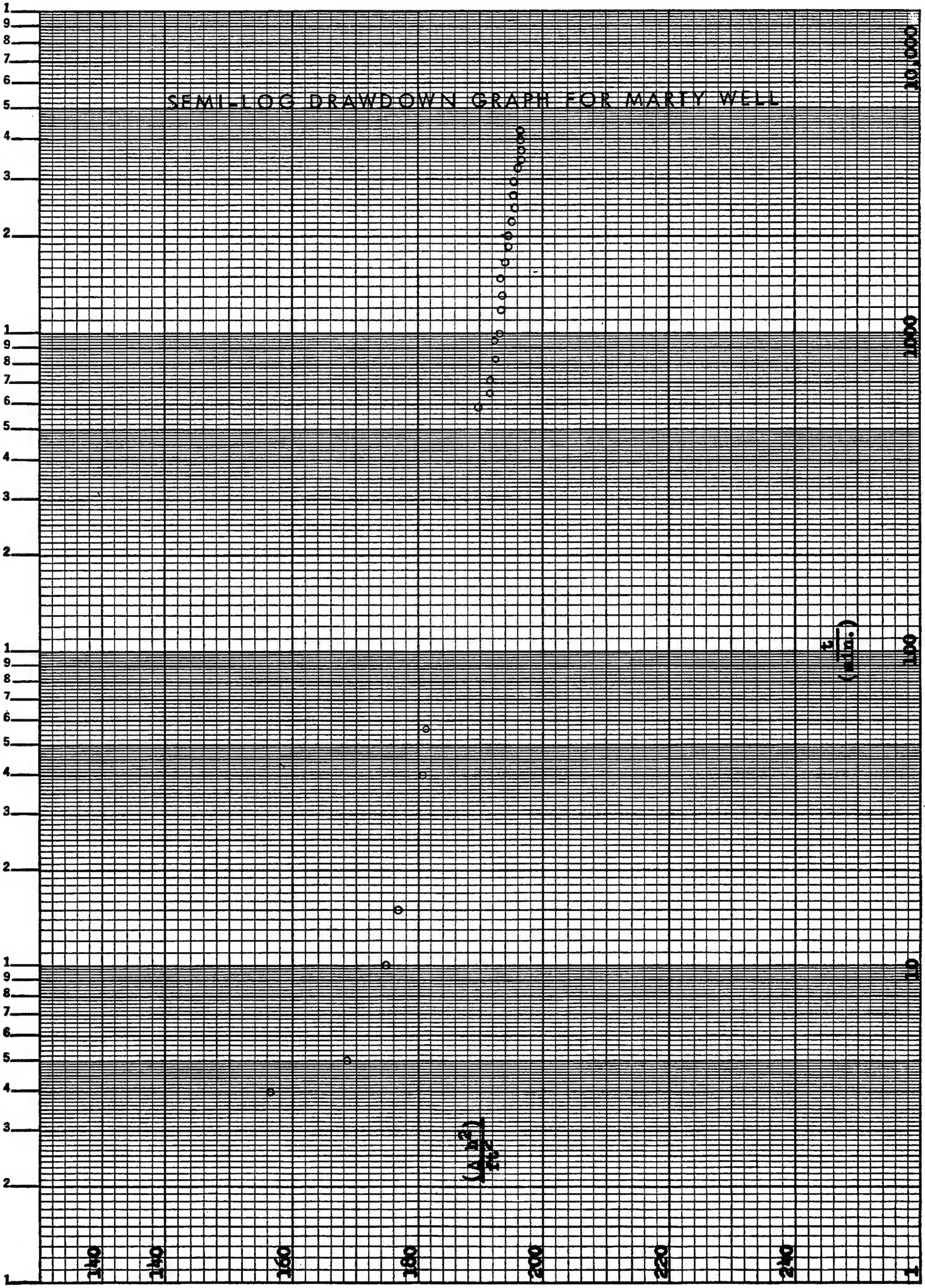

SEMI-LOG DRAWDOWN GRAPH FOR MARTY WELL
t (min.)
10,000
1000
100
10
1
(Δh^2) ft²
140
160
180
200
220
240

FIG. D-57

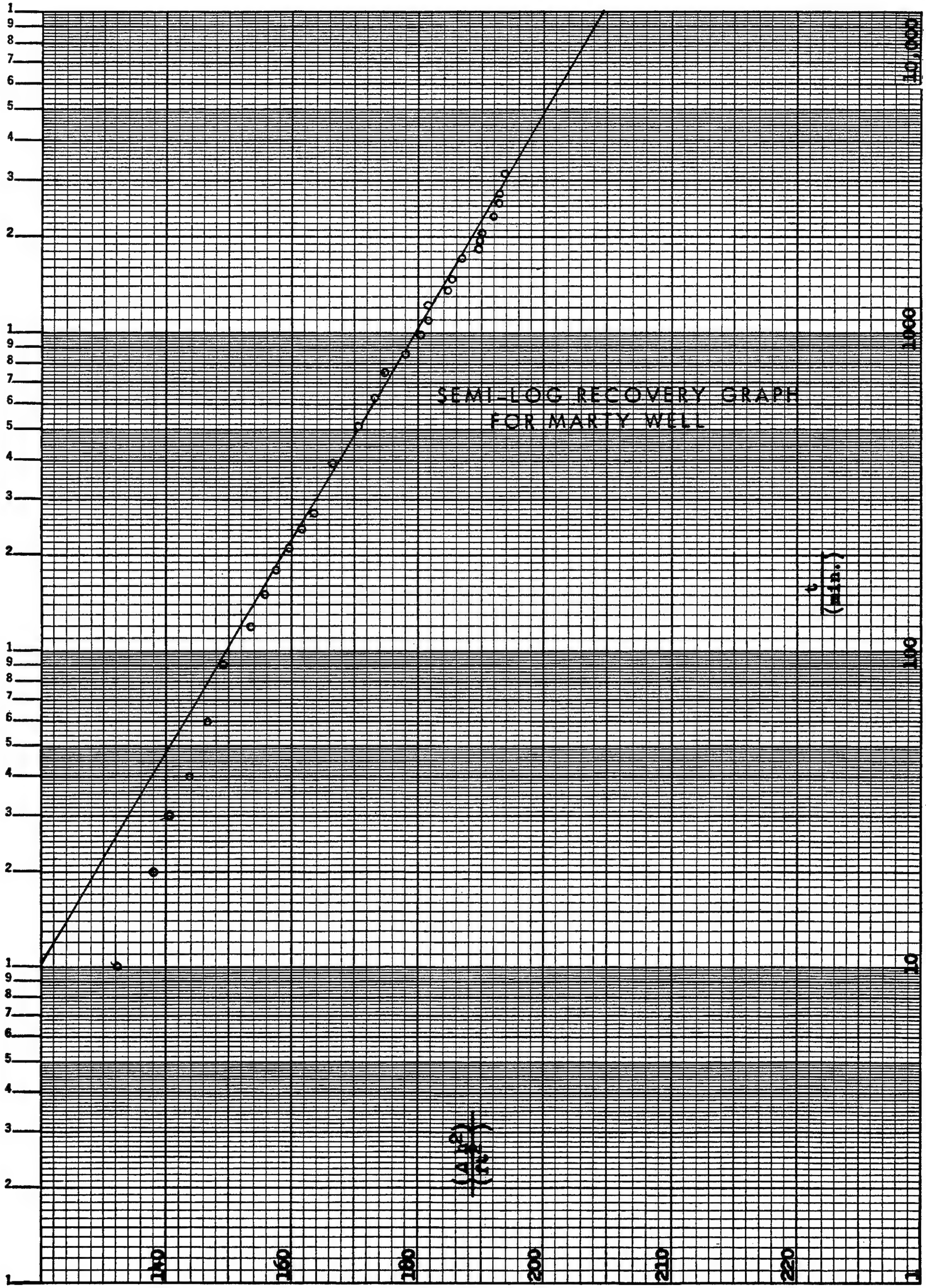
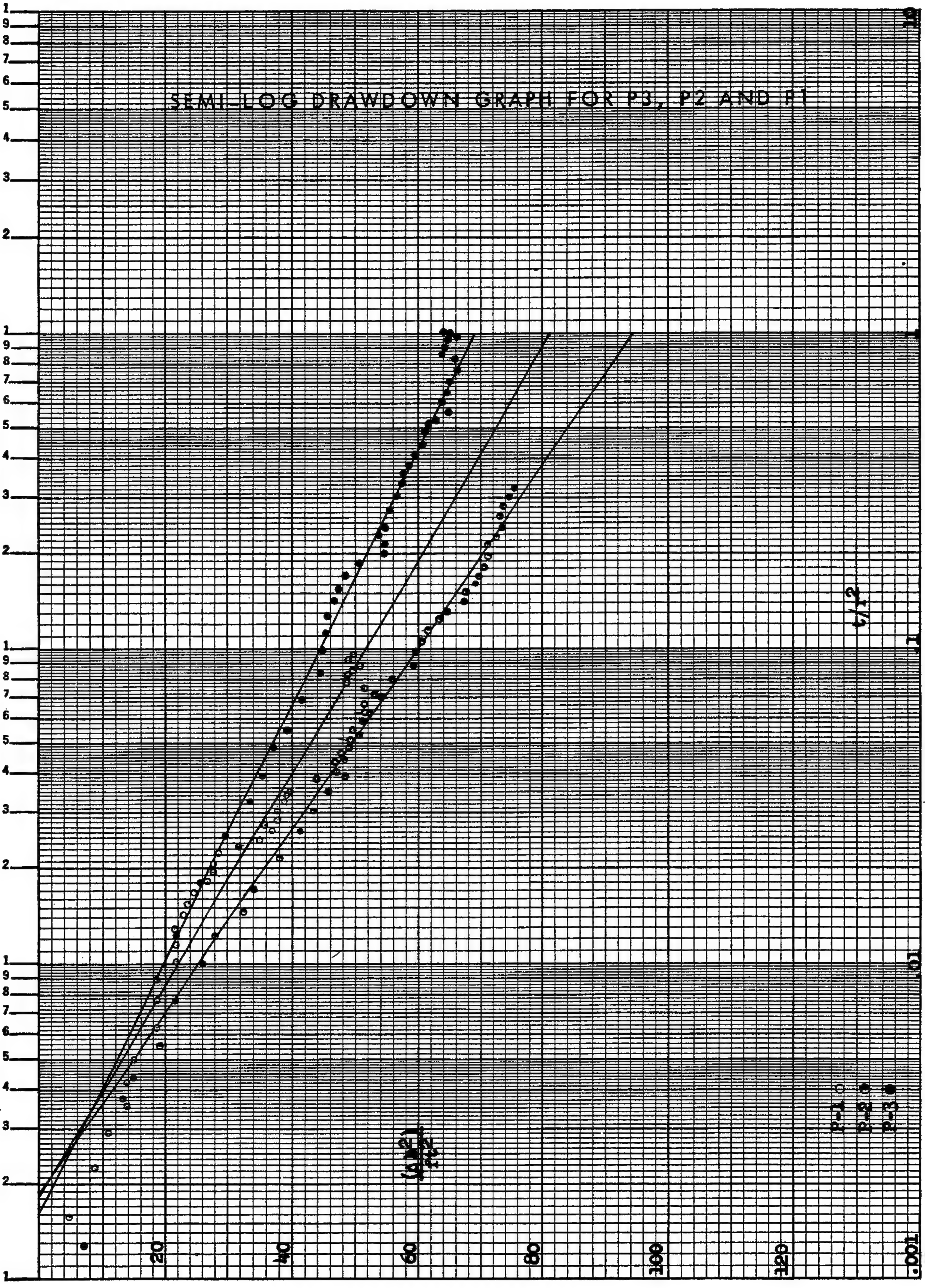
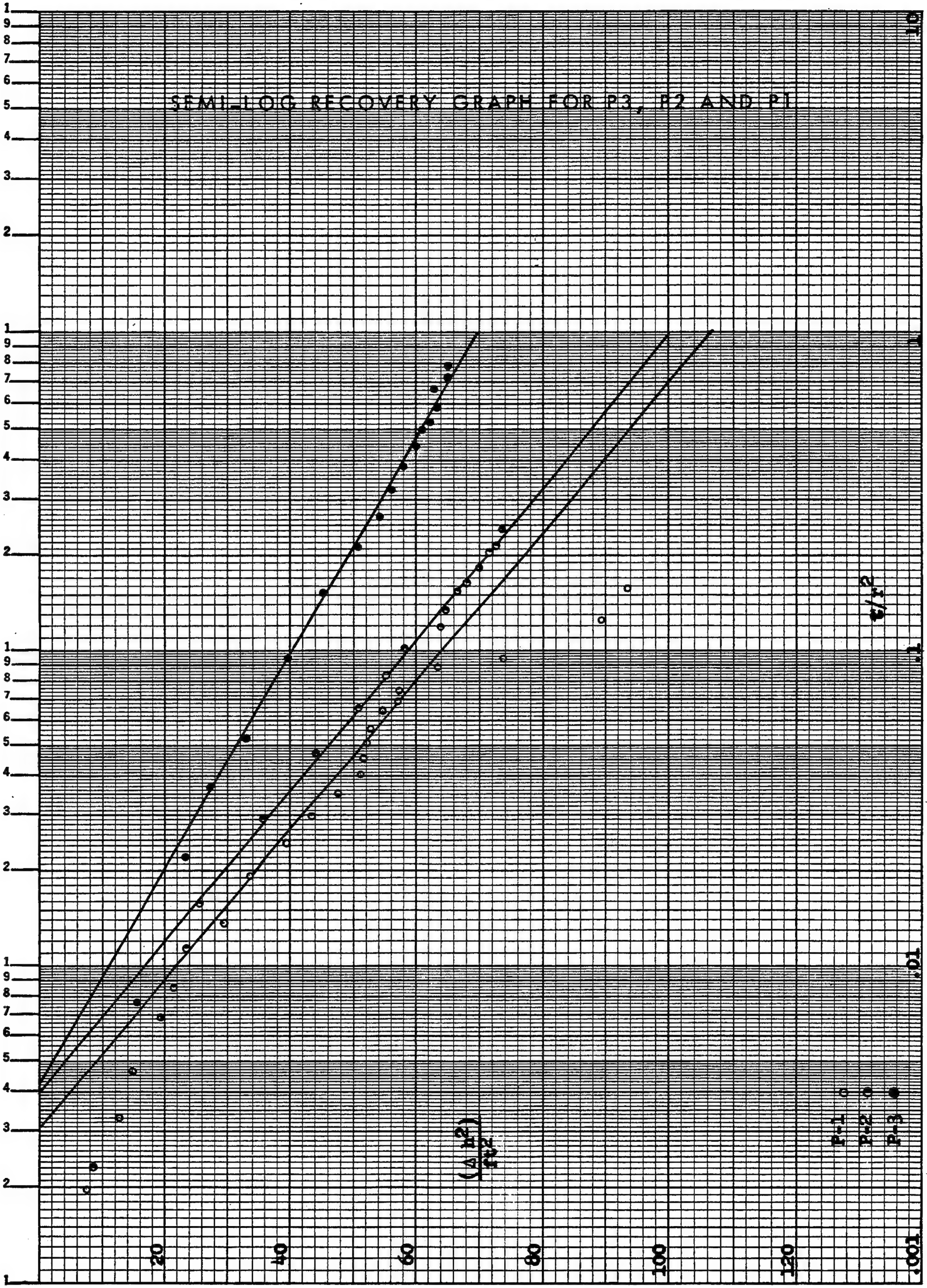

SEMI-LOG RECOVERY GRAPH
FOR MARTY WELL
t (min.)
10,000
1000
100
10
1
140
150
160
170
180
190
200
210
220

FIG. D-58



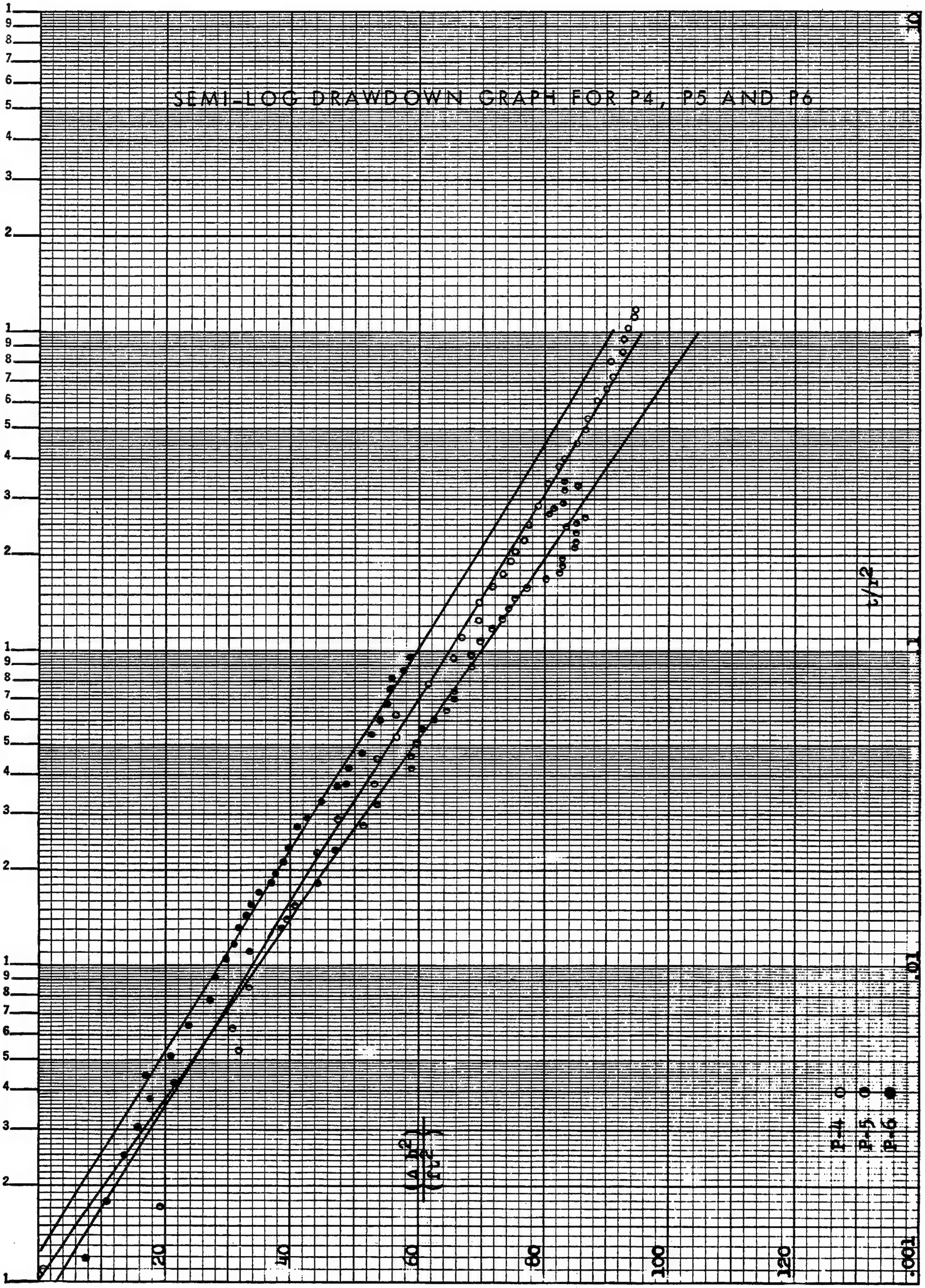
SEMI-LOG DRAWDOWN GRAPH FOR P3, P2 AND P1
t/r²
(Δh²)/2t
P-1
P-2
P-3
20
40
60
80
100
120
.001
.01
.1
1
10

FIG. D-59



SEMI-LOG RECOVERY GRAPH FOR P3, P2 AND P1
$\left(\frac{\Delta h^2}{r^2}\right)$
t/r^2
P-1
P-2
P-3
20
40
60
80
100
120
10
1
.1
.01
.001

FIG. D-60



SEMI-LOG DRAWDOWN GRAPH FOR P4, P5 AND P6
20
40
60
80
100
120
.001
.01
.1
1
10
t/r^2
P-4
P-5
P-6

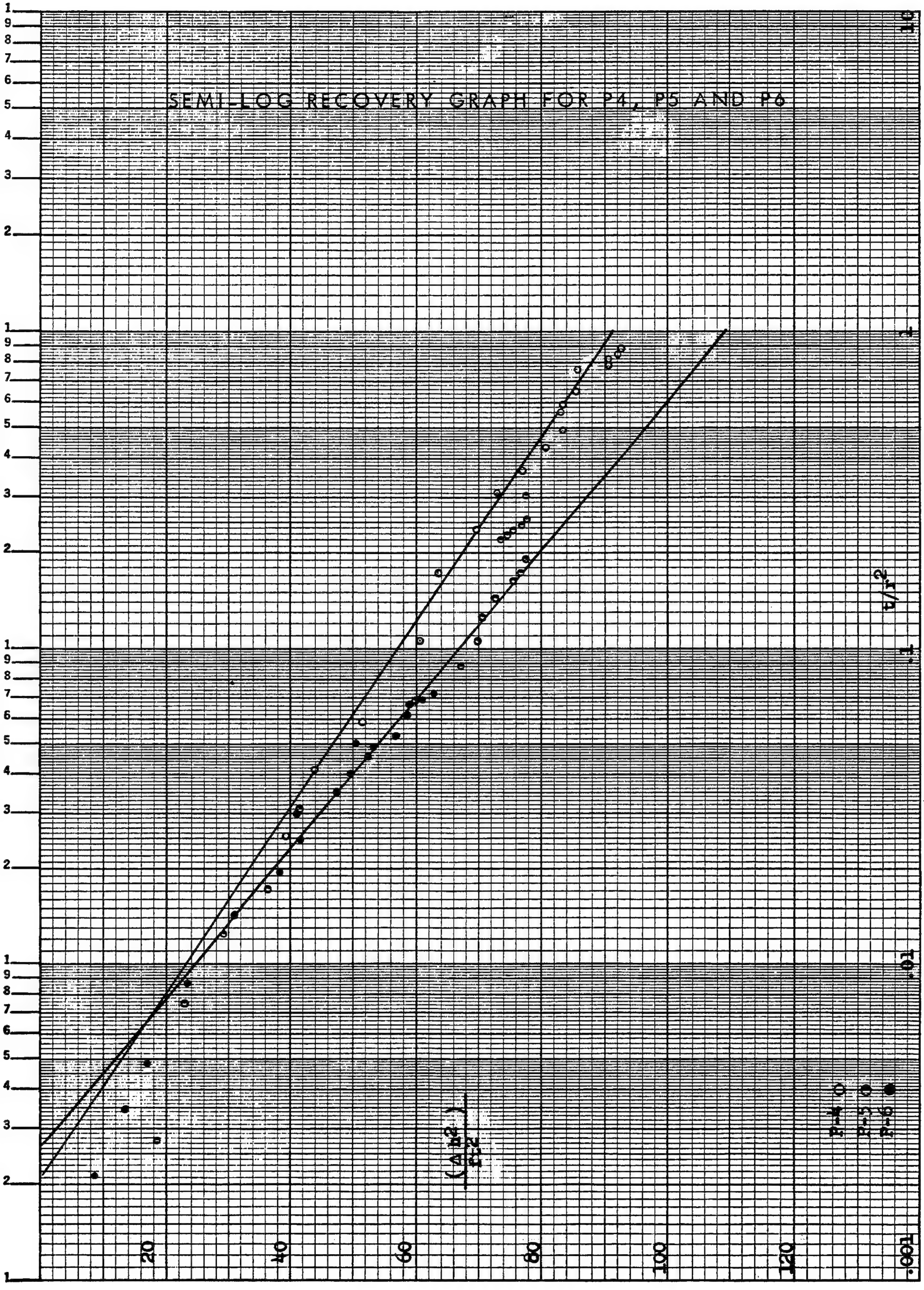

SEMI-LOG RECOVERY GRAPH FOR P4, P5 AND P6
$\left(\frac{\Delta h^2}{t^2}\right)$
t/r^2
P-4
P-5
P-6
20
40
60
80
100
120
10
1
.1
.01
.001

TABLE D-6

DATA FROM INTERFERENCE TEST OF MARTY WELL

[1] Observation well	[2] Distance r/[ft]	[3] Slope $\Delta h^2/[ft^2]$	[4] Hydraulic conductivity K/[ft/day]	[5] Intercept $(t/r^2)_o$ / [min/ft²]	[6] h*/S* / [ft]	[7] h* / [ft]	[8] S*
Marty	--	30 r	1,500	--	--		
P3	64.5	25 d	1,800	0.0016	220	13.8	.063
P2	115.5	34 d	1,330	.0018	260	19.5	.075
P1	213.0	29 d	1,560	.0017	240	22.5	.094
P4	60.8	31 d	1,450	.00081	540	17.5	.033
P5	112.6	35 d	1,290	.00100	500	20.7	.041
P6	212.5	31 d	1,450	.00125	350	21.1	.060
P3	64.5	29 r	1,560	.0043	95	13.8	.145
P2	115.5	42 r	1,070	.0039	150	19.5	.130
P1	213.0	42 r	1,070	.0031	190	22.5	.118
P4	60.8	35 r	1,290	.0021	240	17.5	.073
P5	112.6	43 r	1,050	.0027	225	20.7	.092
P6	212.5	43 r	1,050	.0027	225	21.1	.094
SE Line d			1,560		240		.077
SW Line d			1,400		460		.045
SE Line r			1,230		145		.131
SW Line r			1,130		230		.086
Average			1,330		270		.085

one point of observation this cannot exceed the initial depth of flow. To determine the limits on the apparent storativity, the initial depths of flow are used in place of h* in column 7.

In column 8 values of apparent average storativity are calculated from the figures given in column 6 and 7. These range from 3.3 to 14.5 percent and average about 8.5 percent.

At the bottom of Table D-6 averages are given for each of the two lines of piezometers, both on drawdown and recovery.

It should be noted that the apparent storativity is smaller on both lines for drawdown than it is for recovery.

Interpretation. The storativity remained essentially constant throughout the period of drawdown, and again remained essentially constant though differing in value throughout the subsequent period of recovery. By its constancy and low value the storativity indicates that there may be fine-grain sediments in the upper part of the aquifer overlying the more permeable gravel, in which there is capillary retention of water. An examination of the logs of the wells verifies that this condition exists.

Pumping Test of Nesom Well in 2S67W22

Well 22bcc1 in Section 22 of T2S, R67W was tested by the U. S. Geological Survey in 1955 or 1956. This well is 47.6 feet deep and is cased with 18-inch casing. It was drilled in 1950 and is owned by Harry Nesom. The measuring point for water levels is 0.8 feet above land surface. When measured 13 September 1955, the water level stood 33.67 feet below the measuring point. The measured yield was 305 gpm with a drawdown of 3.8 feet after 10 hours. (See Petri [1956], Table 13.)

By the Thiem Method the transmissivity was found to be about 128,000 gpd/ft, or about 17,000 ft^2/day. They used a saturated thickness of 15 feet and got a hydraulic conductivity ("coefficient of permeability") of 8,500 gpd/ft^2. We calculate 14 feet for the saturated thickness and get about 1,300 ft/day for the hydraulic conductivity.

Test of RMA Well in 3S67W4

Well 4bca1 in Section 4 of T3S, R67W was tested by the Corps of Engineers in January 1952 [U. S. Army, 1953, p 3]. This well was constructed in September 1938 and was cased with 60-inch galvanized casing to a depth of 51 feet and with 48-inch galvanized casing from 51 to 96 feet. The 48-inch casing was gravel packed and slotted. (Petri [1956], Table 13, shows 24 inches for a diameter of this well, but this is evidently an error.)

The well was tested for 44 hours and produced 1,000 gpm with a drawdown of 22.3 feet. During the last six hours the pumping rate was increased to 1,200 gpm and the drawdown was 32.7 feet. Observations of drawdown were made in two observation wells 50 feet away, in one 200 feet away, and in one 300 feet away, but the data were incomplete.

The well was tested again 19 July 1953 for 64 hours and 20 minutes. The discharge rate was 500 gpm. Observations were made of the drawdown in several nearby observation wells. (See USA [1953] Plate 1.) The data were analyzed by the Thiem Method. The average transmissivity was found to be 136,000 gpd/ft or about 18,000 ft^2/day. The storativity was found to be .052. The averages of the results by the Theis graphical method were 130,000 gpd/ft, or 17,000 ft^2/day, for the transmissivity, and .06 for the storativity. As the initial saturated thickness was 45 feet, the hydraulic conductivity was about 380 ft/day.

Characteristics of Wells

<u>RMA Well in 3S 67W4</u>. Using the values of transmissivity and storativity determined from the Corps of Engineers pumping test of this well, and taking the effective radius of the well to be 24 inches or 2.0 feet, the drawdown at the face of the well after one day's continuous pumping can be calculated as follows:

$$s = (2.30\ Q/4\pi T)\log(2.25Tt/Sr_w^2)$$
$$= 0.93\ \text{ft} \times 5.2 = 4.8\ \text{ft}.$$

That is, the drawdown at the face of the well is about 4.8 feet. This may be compared with the drawdown on the graph on Plate 8 of the report by the U. S. Army [1953]. The top inclined line marked "average observed gradient" intersects the vertical line for a distance of 2.0 feet (representing the radius of the well) at 5.5 feet. These two figures are in apparent good agreement, when allowance is made for the difference in elapsed time. However, as indicated in Appendix B under the subsection entitled "Tests in thin confined aquifers," the Theis theory must be modified to apply to h^2 rather than to s. Moreover, allowance much be made for well losses that increase with drawdown.

Discharge-drawdown relation. Figure D-62 is a discharge-drawdown graph for Well 4bcal at Rocky Mountain Arsenal. Recorded thereon are reported measurements of discharge and drawdown. In January 1952 the drawdown was measured at two different rates of discharge. During the pumping test of July 1953 the discharge was measured. The formation loss at that rate of discharge can be inferred from Plate 8 of the Corps of Engineer's report [U. S. Army, 1953]. The U. S. Geological Survey reported a yield of 650 gpm at a (calculated)drawdown of about 21 feet on September 26, 1955 (see Petri [1956, Table 13]).

The bottom of the formation is indicated at a drawdown of 45 feet. The scale on the right hand side of the graph gives the head, or the depth of flow in feet, measured above the bottom of the formation.

The upper curved line in Figure D-62 represents the calculated formation loss at different values of discharge after one day of continuous pumping. The lower curved line drawn through the two plotted points represents an estimate of the discharge-drawdown relationship. It is seen that the maximum capacity of the well is apparently about 1,350 gpm.

The curve giving the combined drawdown, or sum of formation loss and well loss, was determined by assuming that the well-loss coefficient is inversely proportional to the square of the depth of flow. The point obtained on Sept. 26, 1955 falls some distance off the curve. The discharge was only reported, not measured. This may account for part of the discrepancy. The rest may be due to regional drawdown between 1952 and 1955.

Symbols

r = distance from pumping well

r_w = effective radius of pumping well

t = time elapsed since start of pumping

h = depth of unconfined flow at r and t

h^* = effective average depth of flow

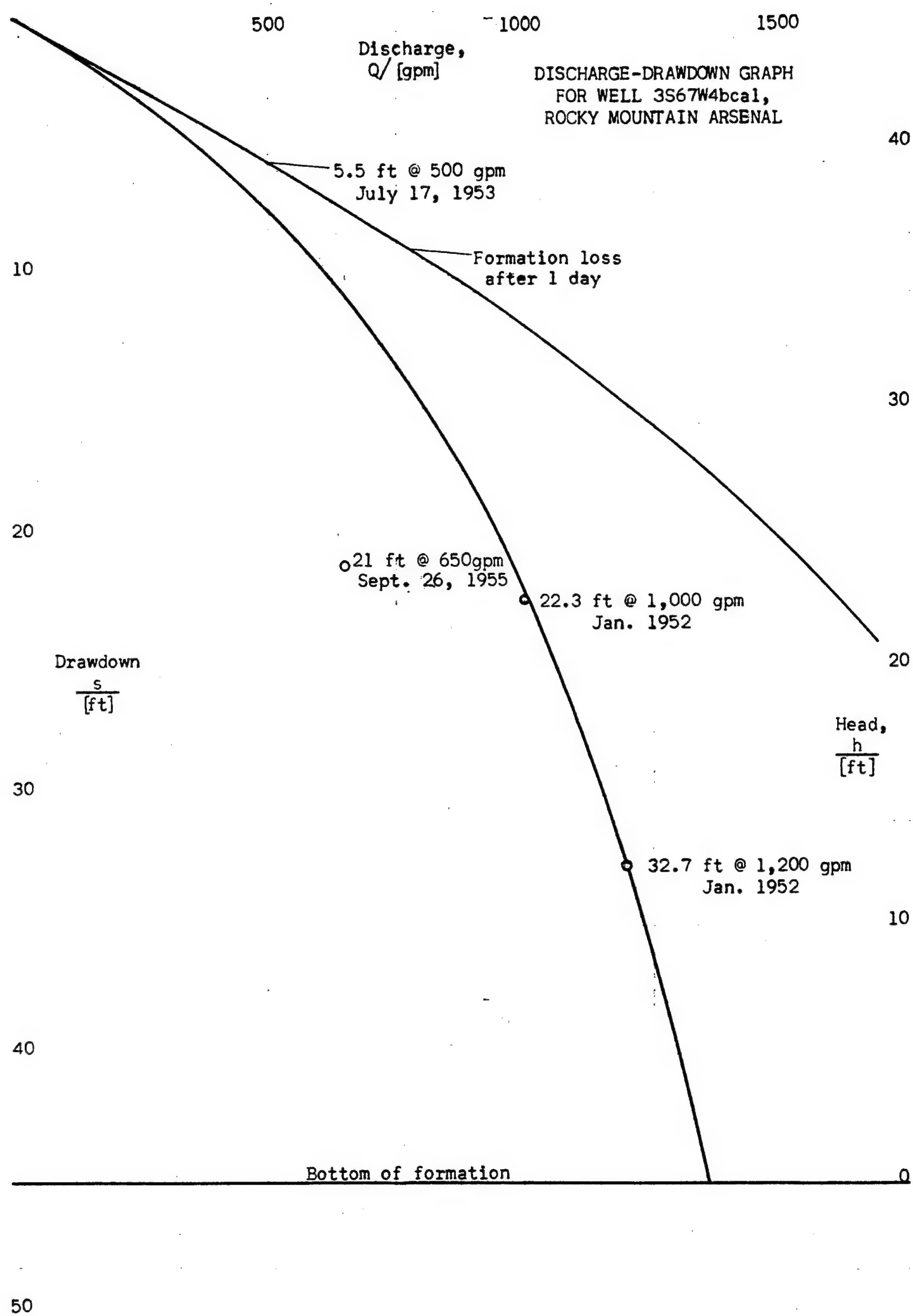
Q = discharge of well

K = hydraulic conductivity of formation

S^* = effective average storativity

C' = well-loss coefficient

FIG. D-62



500
1000
1500
Discharge,
Q/ [gpm]
DISCHARGE-DRAWDOWN GRAPH
FOR WELL 3S67W4bcal,
ROCKY MOUNTAIN ARSENAL
5.5 ft @ 500 gpm
July 17, 1953
Formation loss
after 1 day
21 ft @ 650gpm
Sept. 26, 1955
22.3 ft @ 1,000 gpm
Jan. 1952
32.7 ft @ 1,200 gpm
Jan. 1952
Drawdown
s
[ft]
Head,
h
[ft]
10
20
30
40
50
40
30
20
10
0
Bottom of formation

APPENDIX E

DIFFERENTIAL EQUATIONS FOR NONSTEADY UNCONFINED GROUNDWATER FLOWS AND THEIR DIFFERENCE APPROXIMATIONS

Homogeneous Isotropic Aquifer in Rectangular Coordinates

In rectangular coordinates the second-order partial differential equation for nonsteady unconfined flow of groundwater in a homogeneous isotropic aquifer with a sloping or curved bottom is

$$\frac{\partial}{\partial x}\left[(h-z)\frac{\partial h}{\partial x}\right] + \frac{\partial}{\partial y}\left[(h-z)\frac{\partial h}{\partial y}\right] = \frac{S}{K}\frac{\partial h}{\partial t} \qquad [1]$$

(For the meaning of these and other symbols see the Glossary at the end of the last Appendix.)

In the general case $z = z(x, y)$. That is, z is a function of both x and y.

Equation [1] may be reduced to universal or nondimensional form by putting $x \rightarrow \xi h_o$, $y \rightarrow \eta h_o$, $z \rightarrow \zeta' h_o$ and $h \rightarrow \zeta h_o$, also $t \rightarrow \tau S h_o/K$. Thus,

$$\frac{\partial}{\partial \xi}\left[(\zeta-\zeta')\frac{\partial \zeta}{\partial \xi}\right] + \frac{\partial}{\partial \eta}\left[(\zeta-\zeta')\frac{\partial \zeta}{\partial \eta}\right] = \frac{\partial \zeta}{\partial \tau} \qquad [2]$$

Homogeneous Isotropic Aquifer in Cylindrical Coordinates

In cylindrical coordinates, Equation [1] becomes

$$\frac{1}{r}\frac{\partial}{\partial r}\left[r(h-z)\frac{\partial h}{\partial r}\right] + \frac{\partial}{r\partial \theta}\left[(h-z)\frac{\partial h}{r\partial \theta}\right] = \frac{S}{K}\frac{\partial h}{\partial t} \qquad [3]$$

Putting $r \rightarrow \rho r_w$, $t \rightarrow \tau S r_w^2/K h_o$ and transforming the other symbols as before,

$$\frac{1}{\rho}\frac{\partial}{\partial \rho}\left[\rho(\zeta-\zeta')\frac{\partial \zeta}{\partial \rho}\right] + \frac{\partial}{\rho\partial \theta}\left[(\zeta-\zeta')\frac{\partial \zeta}{\rho\partial \theta}\right] = \frac{\partial \zeta}{\partial \tau} \qquad [4]$$

If the bottom of the aquifer is a sloping plane, we may write $z' = \alpha r \cos\theta$ or $\zeta' = \alpha \rho \cos\theta$, where α is the slope and θ is the azimuth.

If the bottom of the aquifer is a horizontal plane, Equation [4] may be simplified to

$$\frac{1}{\rho^2}\frac{\partial}{\partial \ln\rho}\left[\zeta\frac{\partial \zeta}{\partial \ln\rho}\right] = \frac{\partial \zeta}{\partial \tau} \qquad [5]$$

Putting $\ln \rho = \lambda$ this becomes

$$\frac{e^{-2\lambda}}{2} \cdot \frac{\partial}{\partial \lambda}\left(\frac{\partial \psi^2}{\partial \lambda}\right) = \frac{\partial \psi}{\partial \tau} \qquad [6]$$

which is more convenient for numerical integration.

Inhomogeneous Anisotropic Aquifers

If the aquifer is "orthotropic" as regards hydraulic conductivity, then $K_x \neq K_y$. If it is also inhomogeneous, $\partial K_x / \partial x \neq 0$ and $\partial K_y / \partial y \neq 0$. In this case, therefore,

$$\frac{\partial}{\partial x}\left[K_x (h - z)\frac{\partial h}{\partial x}\right] + \frac{\partial}{\partial y}\left[K_y (h - z)\frac{\partial h}{\partial y}\right] = S \frac{\partial h}{\partial t} \qquad [7]$$

By putting $K_x \rightarrow \kappa_\xi \sqrt{K_x K_y}$ and $K_y \rightarrow \kappa_\eta \sqrt{K_x K_y}$ and transforming the other symbols as before,

$$\frac{\partial}{\partial \xi}\left[\kappa_\xi (\psi - \psi')\frac{\partial \psi}{\partial \xi}\right] + \frac{\partial}{\partial \eta}\left[\kappa_\eta (\psi - \psi')\frac{\partial \psi}{\partial \eta}\right] = \frac{\partial \psi}{\partial \tau} \qquad [8]$$

Homogeneous Anisotropic Aquifers

If the aquifer is orthotropic but homogeneous as regards hydraulic conductivity, then $\partial \kappa_\xi / \partial \xi = 0$ and $\partial \kappa_\eta / \partial \eta = 0$. Putting $\xi \rightarrow \xi' \sqrt{\kappa_\xi}$ and $\eta \rightarrow \eta' \sqrt{\kappa_\eta}$, Equation [8] becomes

$$\frac{\partial}{\partial \xi'}\left[(\psi - \psi')\frac{\partial \psi}{\partial \xi'}\right] + \frac{\partial}{\partial \eta'}\left[(\psi - \psi')\frac{\partial \psi}{\partial \eta'}\right] = \frac{\partial \psi}{\partial \tau} \qquad [9]$$

This transformation preserves the area: $\Delta\xi \, \Delta\eta = \Delta\xi' \Delta\eta'$.

Difference - Differential Equations

The derivation of the difference-differential equations for use in "passive" RC networks (direct analogs) or in DC analog computers (indirect analogs) is best seen by the special case of Equation [2], restricted to one space-coordinate, with $\psi' = 0$:

$$\frac{\partial}{\partial \xi}\left(\psi \frac{\partial \psi}{\partial \xi}\right) = \frac{1}{2}\frac{\partial^2 \psi^2}{\partial \xi^2} = \frac{\partial \psi}{\partial \tau} \qquad [10]$$

We use i as the index of nodes in the mesh, with spacing $\Delta \xi$. Thus Equation [10] becomes

$$\frac{\psi_{i+1}^2 - 2\psi_i^2 + \psi_{i-1}^2}{\overline{\Delta\xi}^2} = \frac{\partial \psi_i}{\partial T} \qquad [11]$$

If there are m nodes, there are m simultaneous non-linear difference-differential equations like [11] to be solved. This the analog computer or the passive network does automatically if the modeling is appropriately carried out.

Difference Equations

For use with a desk calculator or electronic digital computer the equations are differenced both in time and in space. We may use j as the index of points in time and employ the notation $\psi(\xi, T) = \psi(i\Delta\xi, j\Delta T) \longrightarrow \psi(i,j) = \psi_i^j$ using subscripts for space and superscripts for time, understanding that for simplicity either index may be omitted. Thus,

$$\psi_{i+1} = \psi(i+1, j).$$
$$\psi_{i-1} = \psi(i-1, j)$$
$$\psi^{j+1} = \psi(i, j+1)$$

With this notation Equation [10] becomes

$$\Delta\psi^j = \frac{\psi_{i+1}^2 - 2\psi_i^2 + \psi_{i-1}^2}{\overline{\Delta\xi}^2/\Delta T} = \psi^{j+1} - \psi^j \qquad [12]$$

by which the m values of ψ^{j+1} can be calculated from the m known values of ψ^j.

Similar difference approximations may be set up for Equations [2], [4], [6], [8] and [9].

APPENDIX F

UNCONFINED RADIAL FLOW TO WELLS

Single Well in Effectively Infinite Aquifer

The non-linear partial differential equation for nonsteady unconfined flow to a single isolated well has already been given (Equation E-[4]). If the aquifer is effectively infinite, that "boundary condition" may be expressed as follows: $(\partial \zeta^2/\partial \lambda)_\infty = 0$

Boundary Conditions

As the differential equation is of second order in ρ, a second (inner) boundary condition in ρ must be specified to completely define the problem. For example, a "step" well with full instantaneous drawdown would have $\zeta^2 = 0$ for $\lambda = 0$ $(\rho = 1)$ for all positive τ. A "ramp" well might attain full drawdown at a uniform rate during a short initial period of time. A well of constant discharge would have $(\partial \zeta^2/\partial \lambda)_0 =$ const. for all positive τ.

Initial Condition

As the partial differential equation is of first order in τ, one condition on ζ in τ (usually an initial condition) must be specified. In the foregoing examples the initial condition is $\zeta(\lambda) = 0$ at $\tau \leq 0$ for all λ.

Bounded Aquifer

If there is a single well in a bounded horizontal aquifer with no inflow from outside or a large number of wells uniformly spaced on a regular pattern in such an aquifer the outer boundary conditions may be expressed as $\partial \zeta^2/\partial \nu = 0$ on the outer boundary, where ν is the outward directed normal.

Wells in Square Array

An approximate solution for the nonsteady flow to wells in uniform square array may be obtained by replacing each square tributary to a single well by a

circle having the same area, whose radius therefore is $r_e = 2a/\sqrt{\pi}$, $2a$ being the spacing of the wells. An exact solution for steady flow may be obtained in terms of the Green's function for a square. An "exact" solution for nonsteady flow may be obtained only by analog or fine-mesh digital computation.

Wells in Hexagonal Array

Similarly, approximate solutions for the nonsteady flow to wells in uniform hexagonal array may be obtained by replacing each hexagon tributary to a single well by a circle of the same area.

Wells Nonuniformly Spaced

The nonsteady flow to wells unevenly spaced in an unconfined aquifer with a sloping or curved bottom is best handled either by a passive network or by an electronic analog computer. Equation E-[2] may be used, with appropriate modification for locally radial flows and drawdowns at the wells.

Line of Wells

The steady flow of water to a typical well of discharge Q and radius r_w in an infinite line of uniformly spaced wells is given by the equation

$$h^2 = \frac{Q}{2\pi K} \ln \frac{\left(\cosh \frac{\pi y}{2a} - \cos \frac{\pi x}{2a}\right)\left(\cosh \frac{\pi y}{2a} + \cos \frac{\pi x}{2a}\right)}{\pi^2 r_w^2 / 4a^2} + h_w^2 \qquad [1]$$

where h_w is the head at the well-face and $2a$ is the well spacing.

An approximate solution for the nonsteady flow to a line of wells may be obtained in terms of the Green's function for an infinite strip with impermeable boundaries. The "exact" solution may be obtained only by fine-mesh analog or digital computation.

Line of wells cross-gradient. If the line of wells runs cross-gradient and if the initial depth of (inclined) flow is uniform, the flow may be described by solving Equation E-[2] with $\zeta' = \alpha \eta$ and with the conditions on the "impermeable" boundaries or dividing stream-lines $x = \pm a$ appropriately specified: $\partial h/\partial x = 0$. In this case Equation E-[2] is solved by fine-mesh analog or digital computation.

Line of wells down-gradient. If the line of wells runs down-gradient, even if the initial depth of (inclined) flow is uniform, still the flow cannot be divided

by parallel lines as before. In this case the solution is found for the infinite plane by fine-mesh analog or digital computation.

Wells in Square Array in Inclined Uniform Aquifer

With the wells uniformly spaced in square array in an aquifer of initially uniform thickness or depth of flow the flow can be divided by parallel streamlines running down-gradient. Again Equation E-[2] is solved by fine-mesh analog or digital computation with $\zeta' = \alpha\eta$ and with $(\partial h/\partial x) = 0$ at $x = \pm a$.

APPENDIX G

UNCONFINED UNIDIRECTIONAL FLOW TO TRENCHES

Horizontal Aquifer

The differential equation for the nonsteady flow of groundwater to a trench cutting through a thin horizontal unconfined aquifer has already been given (Equation E-[2]). With the trench running in the η-direction the equation becomes

$$\frac{\partial}{\partial \xi}\left(\zeta \frac{\partial \zeta}{\partial \xi}\right) = \frac{1}{2}\frac{\partial^2 \zeta^2}{\partial \xi^2} = \frac{\partial \zeta}{\partial \tau} \qquad [1]$$

The corresponding difference equation is

$$\Delta \zeta^j = \frac{\zeta^2_{i+1} - 2\zeta^2_i + \zeta^2_{i-1}}{2\,\bar{\Delta}\xi^2/\Delta\tau} \qquad [2]$$

For stability we choose $\Delta\tau < \bar{\Delta}\xi^2 / 2$.

Figure G-1 is a logarithmic graph of the water-table decline near a trench with full drawdown in an aquifer with horizontal bottom and initially horizontal water-table. The plotted points were obtained by numerically integrating Equation [2] using a desk calculator. Only a limited range of the composite variable $\xi/2\sqrt{\tau}$ is covered. In order to calculate the variation of flow into the trench, a much longer run on a digital computer with finer mesh is needed.

Inclined Aquifer

The differential equation for the nonsteady flow of groundwater to a trench cutting through a thin inclined unconfined aquifer has also been given (Equation E-[2], with $\zeta' = \alpha\xi = \xi'$). Putting $\alpha^2\tau \rightarrow \tau'$ we get

$$\frac{\partial}{\partial \xi'}\left[(\zeta - \xi')\frac{\partial \zeta}{\partial \xi'}\right] = \frac{\partial \zeta}{\partial \tau'} \qquad [3]$$

Water-table decline upstream. Figure G-2 is a logarithmic graph of the water-table decline upstream from a trench with full drawdown in an aquifer with inclined plane bottom and initially parallel inclined water table. The

plotted points for different values of ξ' were obtained by numerically integrating Equation [3] using a desk calculator. Again, only a limited range of the composite variable $\xi'/2\sqrt{\tau'}$ is covered. Digital calculations with electronic computer are needed to cover the full range and accurately calculate the inflow into the trench from upstream.

Water-table decline downstream. Figure G-3 is a similar logarithmic graph of the water-table decline downstream from such a trench. The decline is much more pronounced downstream than upstream.

Figures G-1, 2 and 3 are merely illustrative. Nearly instantaneous draw-down was applied at the face of the trench. Other, more realistic examples must be run to elucidate the behavior of such a non-linear system.

APPENDIX H

SEMI-CONFINED FLOWS AND MIXED FLOWS

Transmissive Aquifer with Storative Semi-Confining Layer

Assume a relatively transmissive aquifer of thickness b and hydraulic conductivity K is overlain by a much less transmissive but more storative confining bed of hydraulic conductivity K' and storativity S'. The flow may be considered to be completely vertical in the confining bed and completely horizontal in the aquifer, the refraction being virtually $90°$ at their interface.

The vertical flow in the confining bed is governed by the equation

$$K' \frac{h'-h}{h'} = -S' \frac{\partial h'}{\partial t} \qquad [1]$$

where h is the head in the aquifer and h' is the elevation of the water table in the confining bed.

The horizontal flow in the aquifer is governed by

$$\frac{Kb}{r} \frac{\partial}{\partial r}\left(r \frac{\partial h}{\partial r}\right) = -K' \frac{h'-h}{h'} \qquad [2]$$

Equations [1] and [2] share a common term, which relates to the interflow between the confining layer and the aquifer.

By an appropriate reduction to nondimensional form Equations [1] and [2] become

$$\frac{\partial \eta'}{\partial \tau} = -\frac{\eta'-\eta}{\eta'} = e^{-2\lambda}\, \partial^2 \eta / \partial \lambda^2 \qquad [3]$$

with $\lambda = \ln \rho$.

Equations [3] are amenable to solution by digital computation, solving the right-hand equation first and the left-hand equation afterward in successive alternations. Their equivalents in differenced form are:

$$\frac{\Delta \eta'}{\Delta \tau} = \frac{\eta_i' - \eta_i}{\eta_i'} = \frac{e^{-2\lambda_i}}{\Delta \lambda^2}\left(\eta_{i+1} - 2\eta_i + \eta_{i-1}\right) \qquad [4]$$

For a steady well in an effectively infinite aquifer the inner boundary conditions are $(\partial \eta / \partial \lambda)_0 =$ const., $(\partial \eta' / \partial \lambda)_0 = 0$ and the outer boundary conditons are $(\partial \eta / \partial \lambda)_\infty = 0 = (\partial \eta' / \partial \lambda)_\infty$. The initial condition might be $\eta(\rho) = \eta'(\rho) =$ const.

Mixed Flows

Of great practical interest in the present investigation is the case where a radial flow may initially be confined or semi-confined uniformly over all ρ and where unconfinement may develop at the well and spread outward from the well. From the well-face (*at* $r = r_w$) out to some moving cylindrical boundary (*at* $r = r_*$) the flow may be governed by Equation E-[6] while beyond that moving boundary it may be governed by Equation [3].

Crude attempts have been made to solve this system of equations with little success. Computation by analog or digital methods will give satisfactory results.

APPENDIX I

DISPERSION OF CONTAMINANTS IN GROUNDWATER

Unidirectional Dispersion

Dispersion in a uniform unidirectional flow of groundwater is governed by the equation

$$D \frac{\partial^2 c}{\partial x^2} - \frac{\partial c}{\partial x} = \frac{1}{V} \frac{\partial c}{\partial t} \qquad [1]$$

where D is the dispersion constant, c is the concentration, and v is the average velocity ($= Q / Af$, with f = porosity).

The difference equation corresponding to Equation [1] is

$$D \frac{c_{i+1} - 2c_i + c_{i-1}}{\overline{\Delta x}^2} - \frac{c_{i+1} - c_{i-1}}{2\Delta x} = \frac{1}{V_i} \frac{\Delta c_i}{\Delta t} \qquad [2]$$

This is amenable to analysis and has a solution in terms of the error function.

Radial Dispersion

Dispersion in a steady radial flow of strength Q in an aquifer of thickness b is governed by

$$D \frac{\partial^2 c}{\partial r^2} - \frac{\partial c}{\partial r} = \frac{2\pi r b f}{Q} \frac{\partial c}{\partial t} \qquad [3]$$

This equation, or rather its differenced equivalent, is not amenable to exact mathematical analysis but should be solved by analog or digital computation.

Stratified Groundwater Flow

It is important to understand the theory of stratified flow in unconfined groundwater systems. Consider the flow of fresh water of viscosity μ and specific weight γ beneath a water table whose elevation is h in an aquifer of porosity f and permeability k. Let this flow be bounded beneath by its interface with a parallel or opposing flow of salt water of viscosity μ' and specific weight γ'. The elevation of the interface is z, and the elevation of the bottom of the aquifer is z'.

By equating pressures at the interface at elevation z it is found that for "gradually varied" flow $\gamma' h' = \gamma h + (\gamma' - \gamma) z$, where h' is the head (assumed uniform over the vertical) in the salt water.

The fresh-water flow is governed by the equation

$$S \frac{\partial h}{\partial t} - \frac{\gamma k}{\mu} \frac{\partial}{\partial x}\left[(h - z)\frac{\partial h}{\partial x}\right] = f \frac{\partial z}{\partial t} \qquad [4]$$

Compare this equation with Equation E-[1].

The salt-water flow is governed by

$$\frac{\gamma' k}{\mu'} \frac{\partial}{\partial x}\left[(z - z')\frac{\partial h'}{\partial x}\right] = f \frac{\partial z}{\partial t} \qquad [5]$$

Using the relation between h' and h and z and simplifying, Equations [4] and [5] may be written

$$\frac{S}{K}\frac{\partial h}{\partial t} - \frac{\partial}{\partial x}\left[(h - z)\frac{\partial h}{\partial x}\right] = \frac{f}{K}\frac{\partial z}{\partial t} = \frac{\mu}{\mu'}\frac{\partial}{\partial x}\left[(z - z')\frac{\partial}{\partial x}\left(h - \frac{\gamma' - \gamma}{\gamma} z\right)\right] \qquad [6]$$

By appropriate reduction to nondimensional form Equations [6] become

$$\frac{\partial \eta}{\partial T} - \frac{\partial}{\partial \xi}\left[(\eta - \eta')\frac{\partial \eta}{\partial \xi}\right] = \frac{\partial \eta'}{\partial T} = \frac{\mu}{\mu'}\frac{\partial}{\partial \xi}\left[(\eta' - \eta'')\frac{\partial}{\partial \xi}\left(\eta - \frac{\gamma' - \gamma}{\gamma}\eta'\right)\right] \qquad [7]$$

Equations [7] are amenable to digital computation by solving the left-hand equation first and the right-hand equation afterward, in successive alternations. Or, they may be solved by electronic analog.

Dispersion in Stratified Flows

The dispersion of ions in stratified-flow systems may be described by appropriate combinations of the concepts formalized under the three preceding topics. Also allowance may be made for the influence of inhomogeneity and anisotropy by incorporating the principles of Equations E-[8] and E-[9].

GLOSSARY

Listed and explained below are the more important mathematical symbols employed in the appendices, particularly in Appendices E through I. For other symbols see page D-21.

a = half-width of flow-strip in horizontal plane, or other characteristic horizontal length.

b = thickness of an aquifer.

c = concentration of an ion subject to dispersion.

D = dispersion constant of a porous medium such as an aquifer.

f = porosity of aquifer.

h = head, or elevation of water table above arbitrary datum (datum is usually bottom of aquifer in horizontal case). Also, depth of unconfined flow.

h_o = head at $x = o$ or at $r = o$ (origin of space coordinates).

h° = initial head, i.e. at $t = o$ (origin of time coordinates).

i = index for nodes in space-mesh or network.

j = index for point in time in difference equation.

K = hydraulic conductivity, the product of permeability by specific weight of fluid, divided by viscosity of fluid. Commonly miscalled "permeability" and "coefficient of permeability."

K_x = x-component of hydraulic-conductivity tensor.

K_y = y-component of hydraulic-conductivity tensor.

k = permeability

Q = discharge of well

S = storativity or "coefficient of storage" (Theis)

t = time (usually time elapsed from start of pumping or start of draining).

V = average velocity of groundwater flow ($= Q/Af$)

x = horizontal coordinate, normal to y

y = horizontal coordinate, normal to x

z = elevation of bottom of aquifer above arbitrary datum, $z = z(x, y)$

α = angle or slope of plane bottom of aquifer.

γ = specific weight (weight per unit volume) of fluid such as water.

ζ = nondimensional vertical space coordinate, or head-ratio.

η = nondimensional horizontal space coordinate, normal to ξ.

$\eta = y/h_o$ or y/a.

θ = azimuthal angle.

κ_ξ = ξ -component of conductivity-ratio tensor.

κ_η = η -component of conductivity-ratio tensor.

λ = $\ln \rho$ (natural logarithm)

μ = viscosity

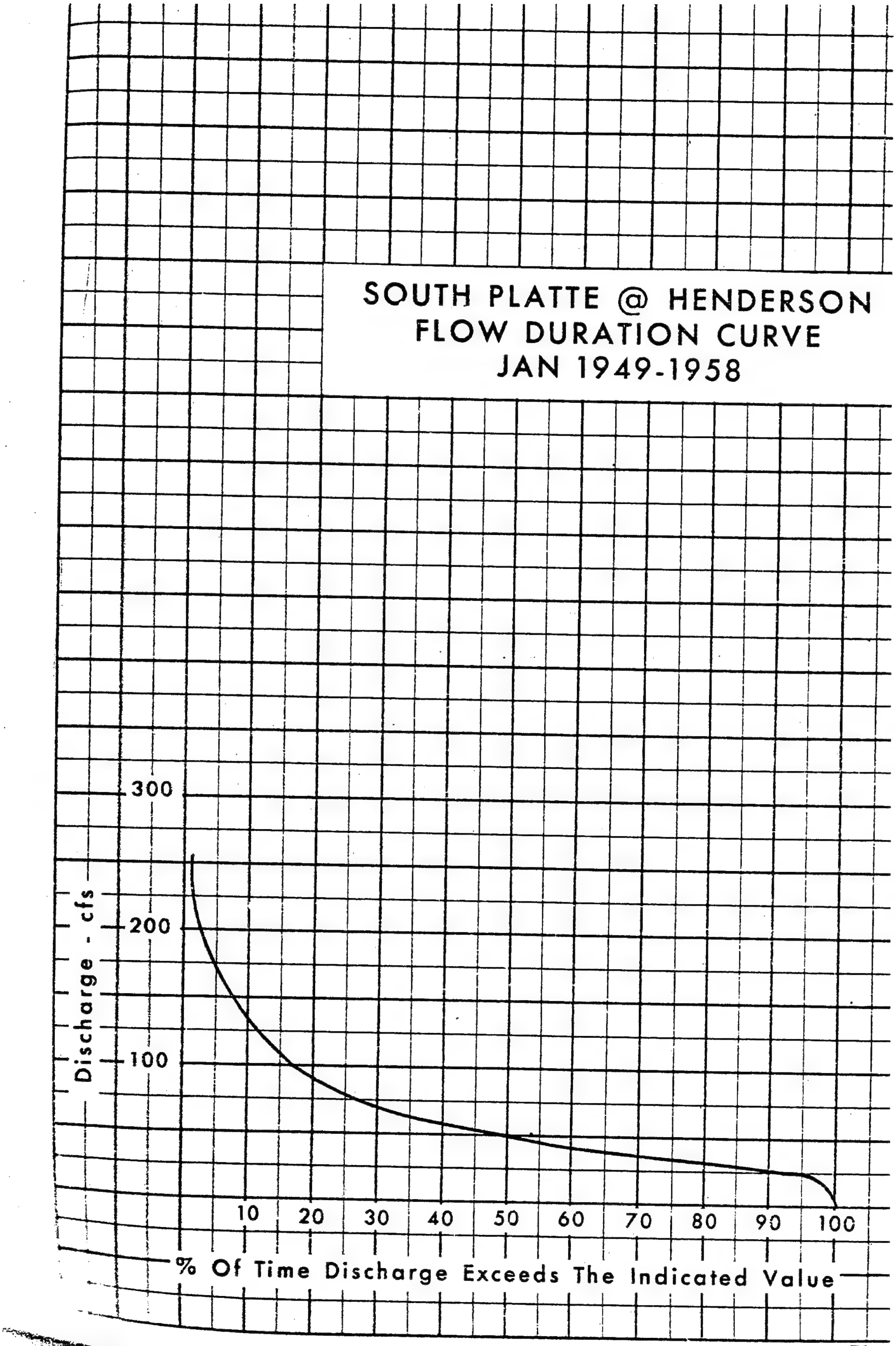
ν = nondimensional coordinate, normal to boundary

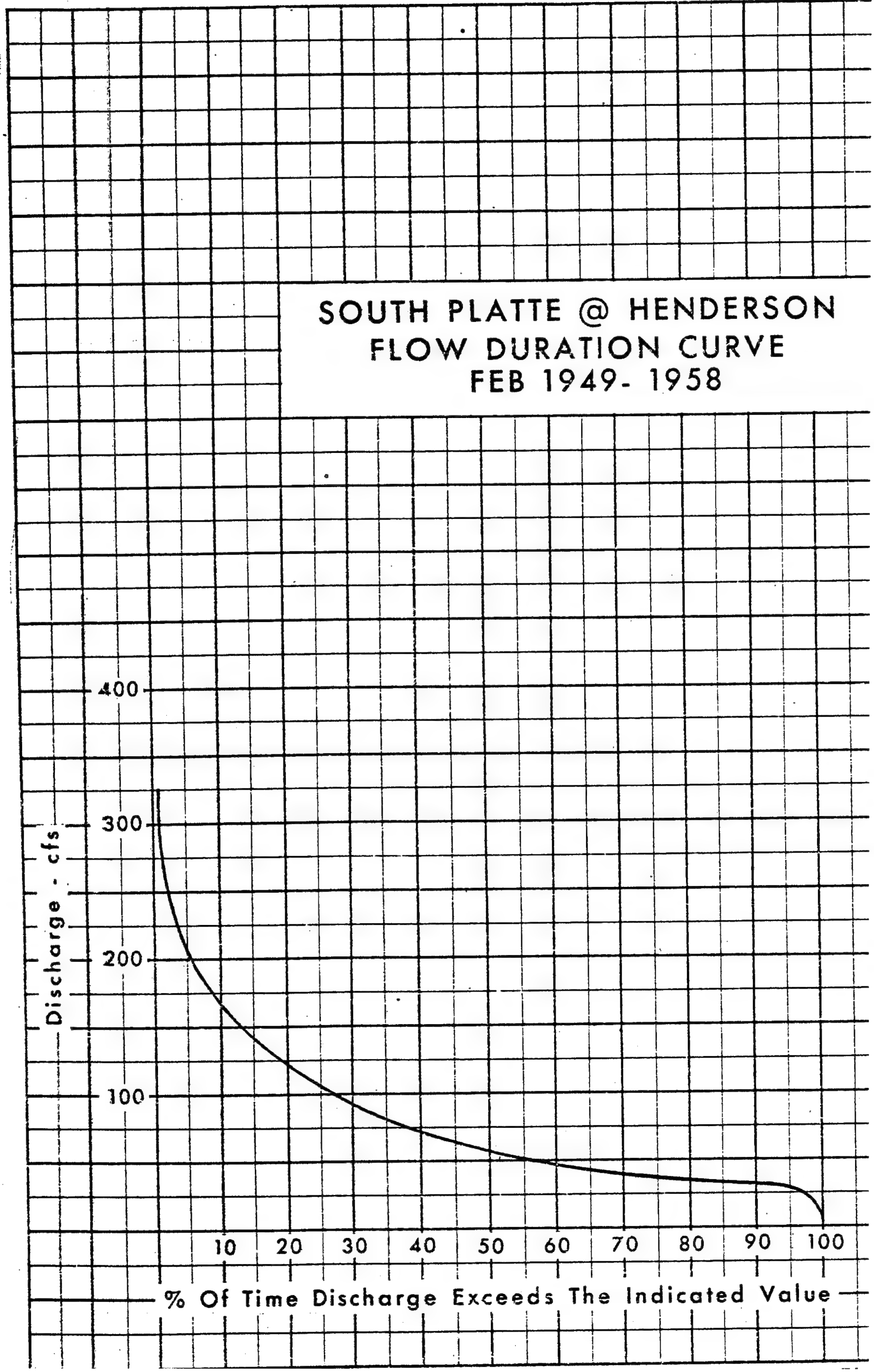
ξ = nondimensional horizontal space coordinate, normal to η. $\xi = x/h_o$ or x/a.

ρ = nondimensional radial coordinate $\rho = r/r_w$.

τ = nondimensional time coordinate, or time-ratio.

$\tau = Kt/Sh_o$ or Kh_ot/Sa^2, depending on the case.


SOUTH PLATTE @ HENDERSON
FLOW DURATION CURVE
JAN 1949-1958
300
200
100
Discharge - cfs
10
20
30
40
50
60
70
80
90
100
% Of Time Discharge Exceeds The Indicated Value
Fig.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FLOW DURATION CURVE
FEB 1949- 1958
400
300
200
100
Discharge - cfs
10
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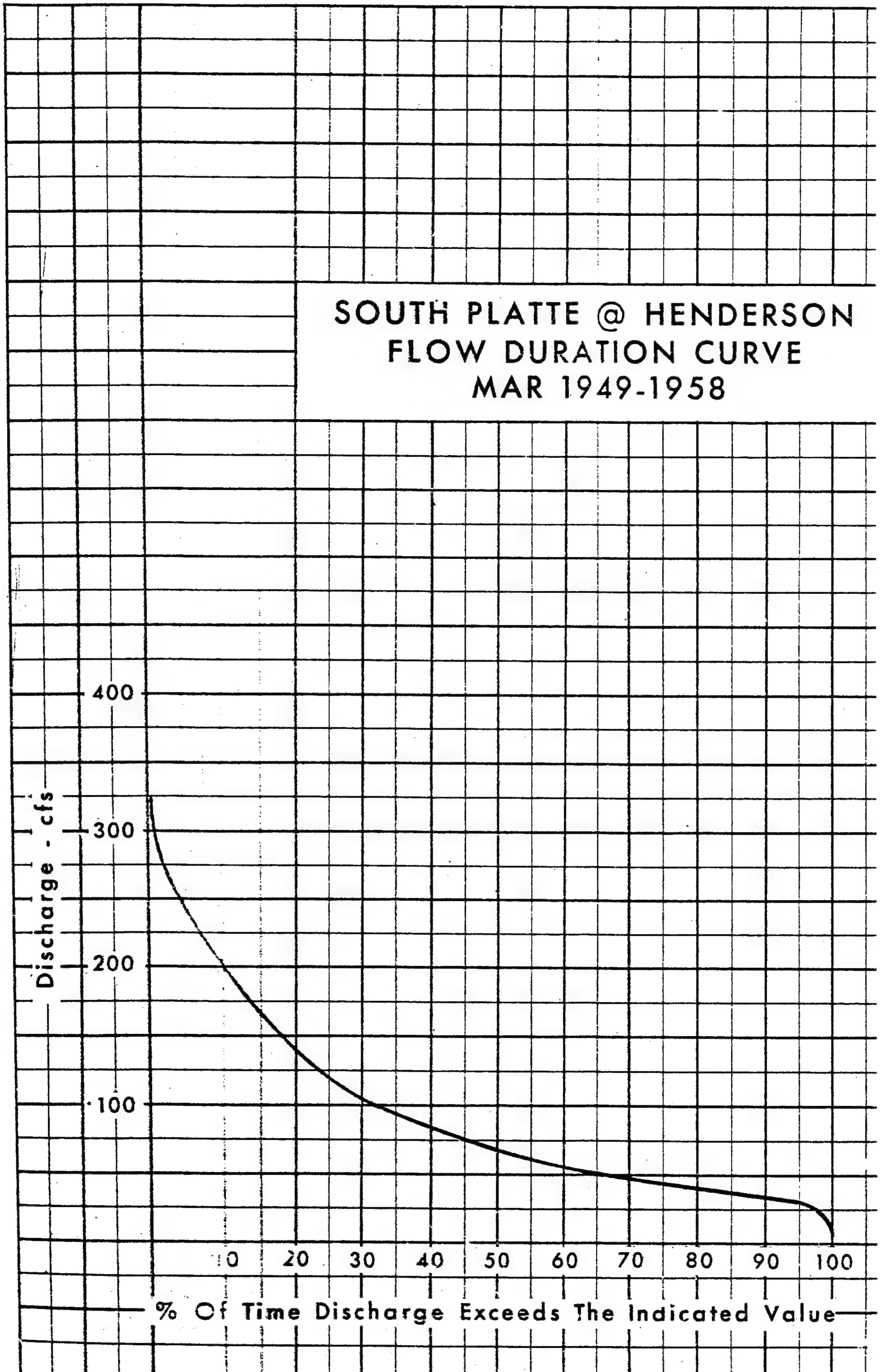
100
% Of Time Discharge Exceeds The Indicated Value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FLOW DURATION CURVE
MAR 1949-1958
Discharge - cfs
400
300
200
100
10
20
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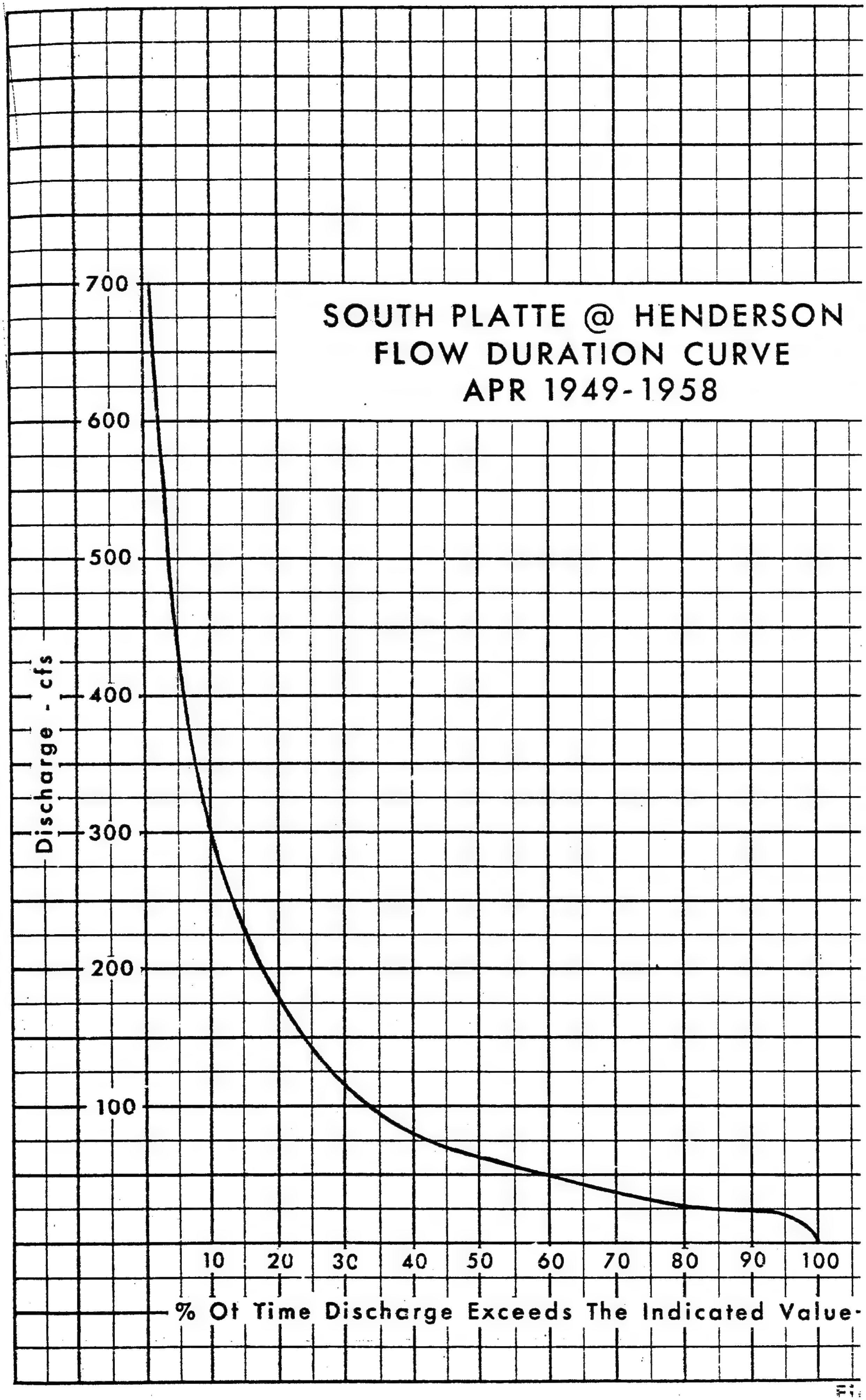
50
60
70
80
90
100
% Of Time Discharge Exceeds The Indicated Value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FLOW DURATION CURVE
APR 1949-1958
Discharge - cfs
700
600
500
400
300
200
100
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20
30
40
50
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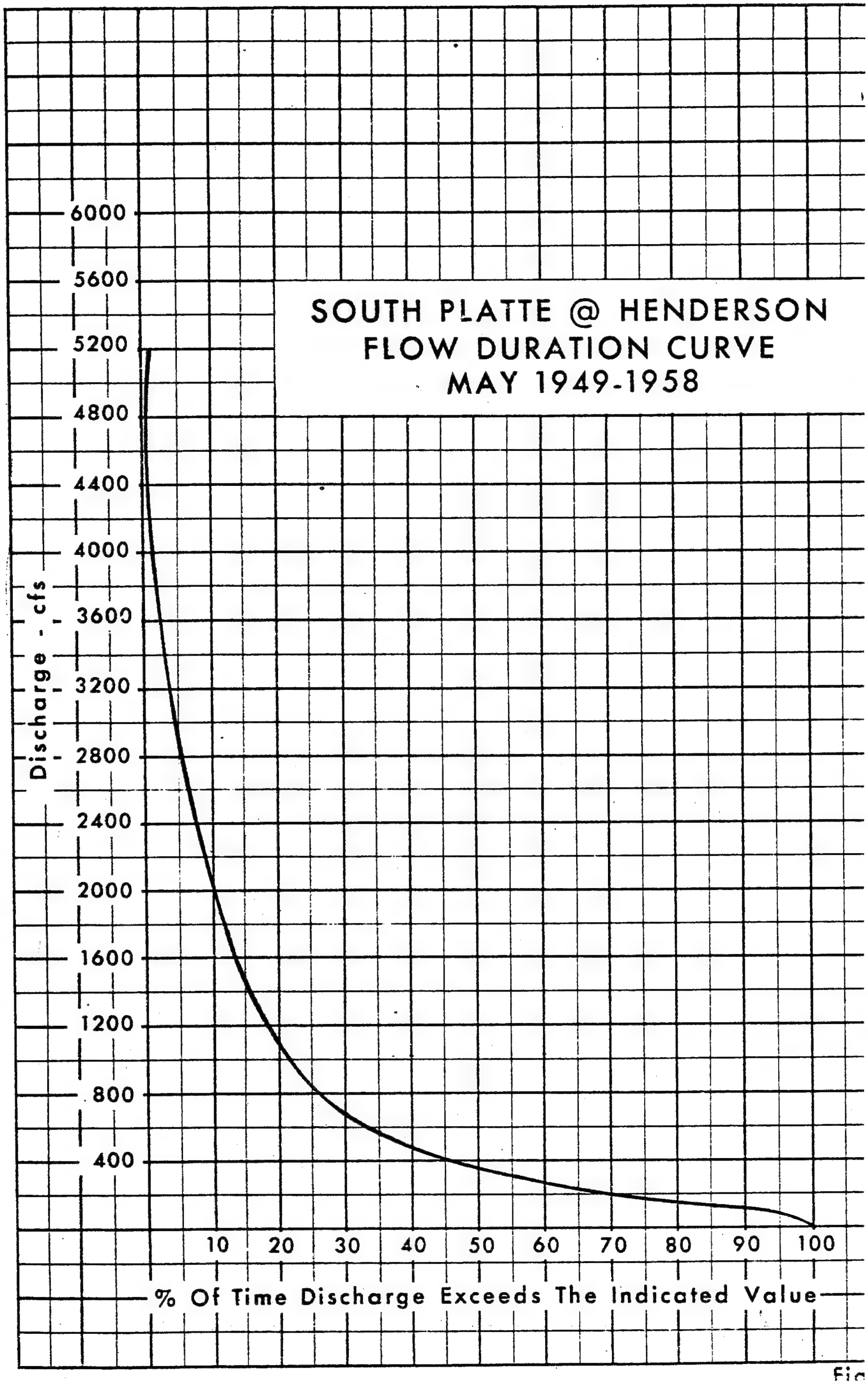
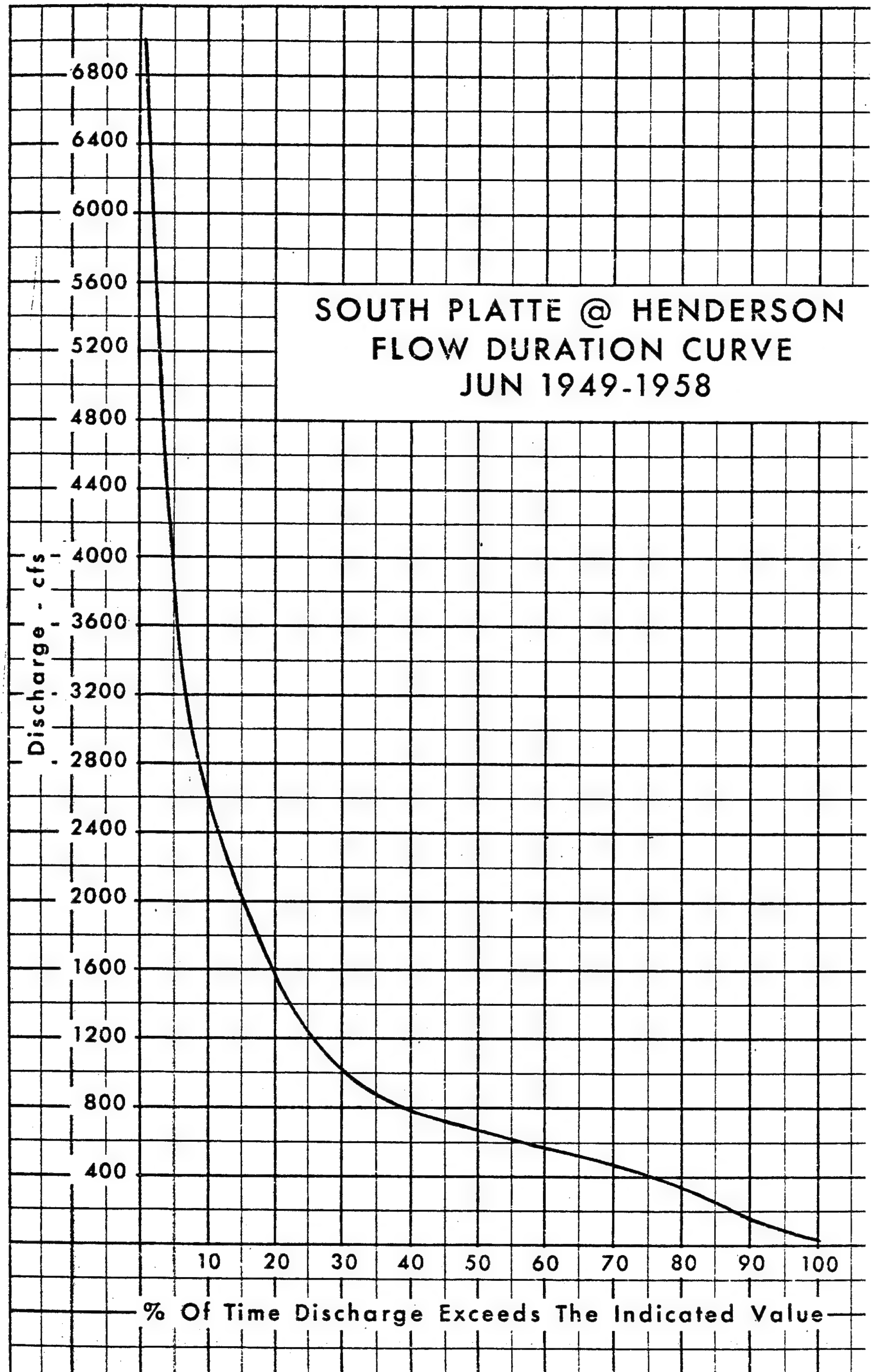
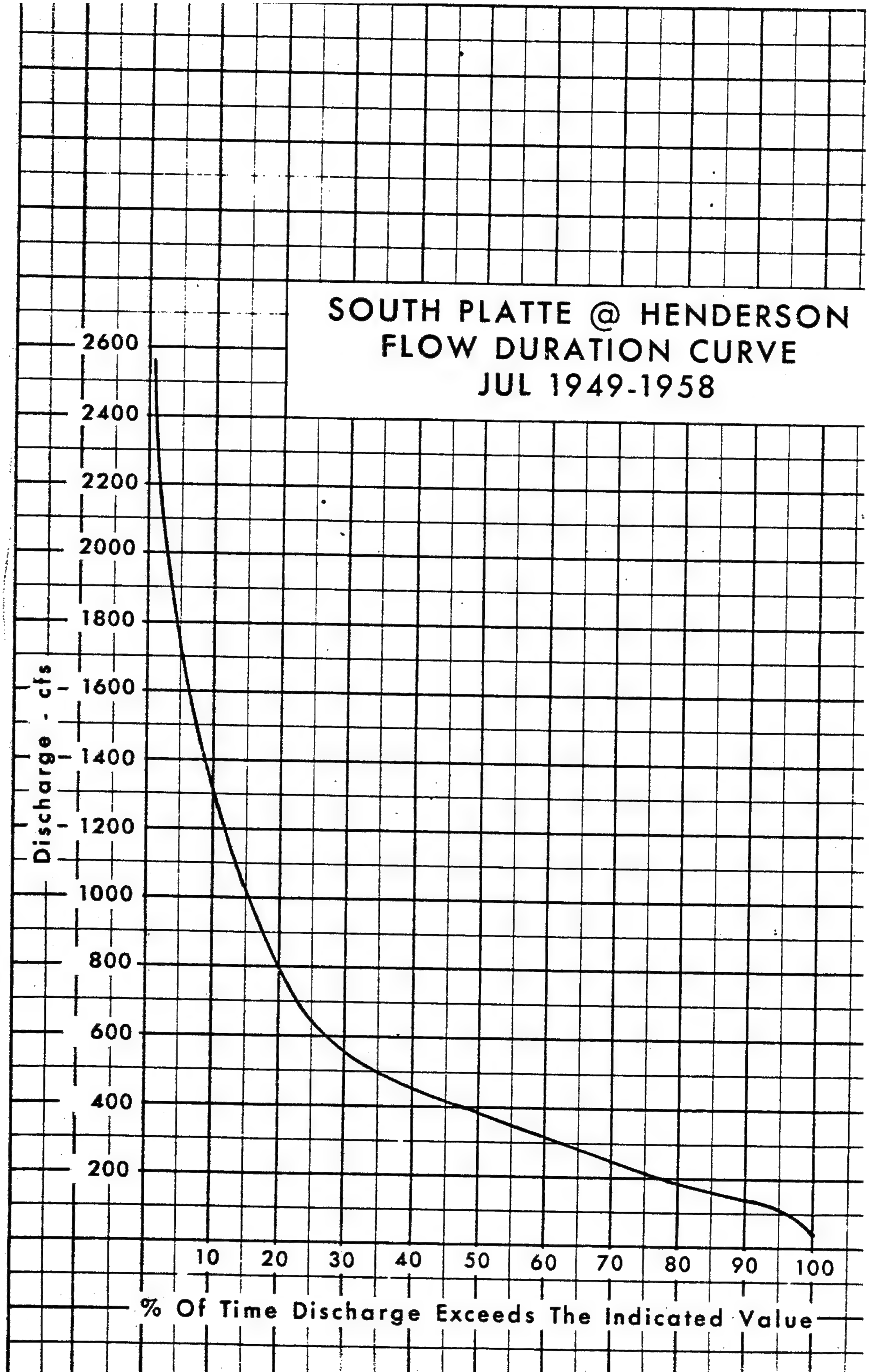
70
80
90
100
% Of Time Discharge Exceeds The Indicated Value


SOUTH PLATTE @ HENDERSON
FLOW DURATION CURVE
MAY 1949-1958
Discharge - cfs
6000
5600
5200
4800
4400
4000
3600
3200
2800
2400
2000
1600
1200
800
400
10
20
30
40
50
60
70
80
90
100
% Of Time Discharge Exceeds The Indicated Value

Fig

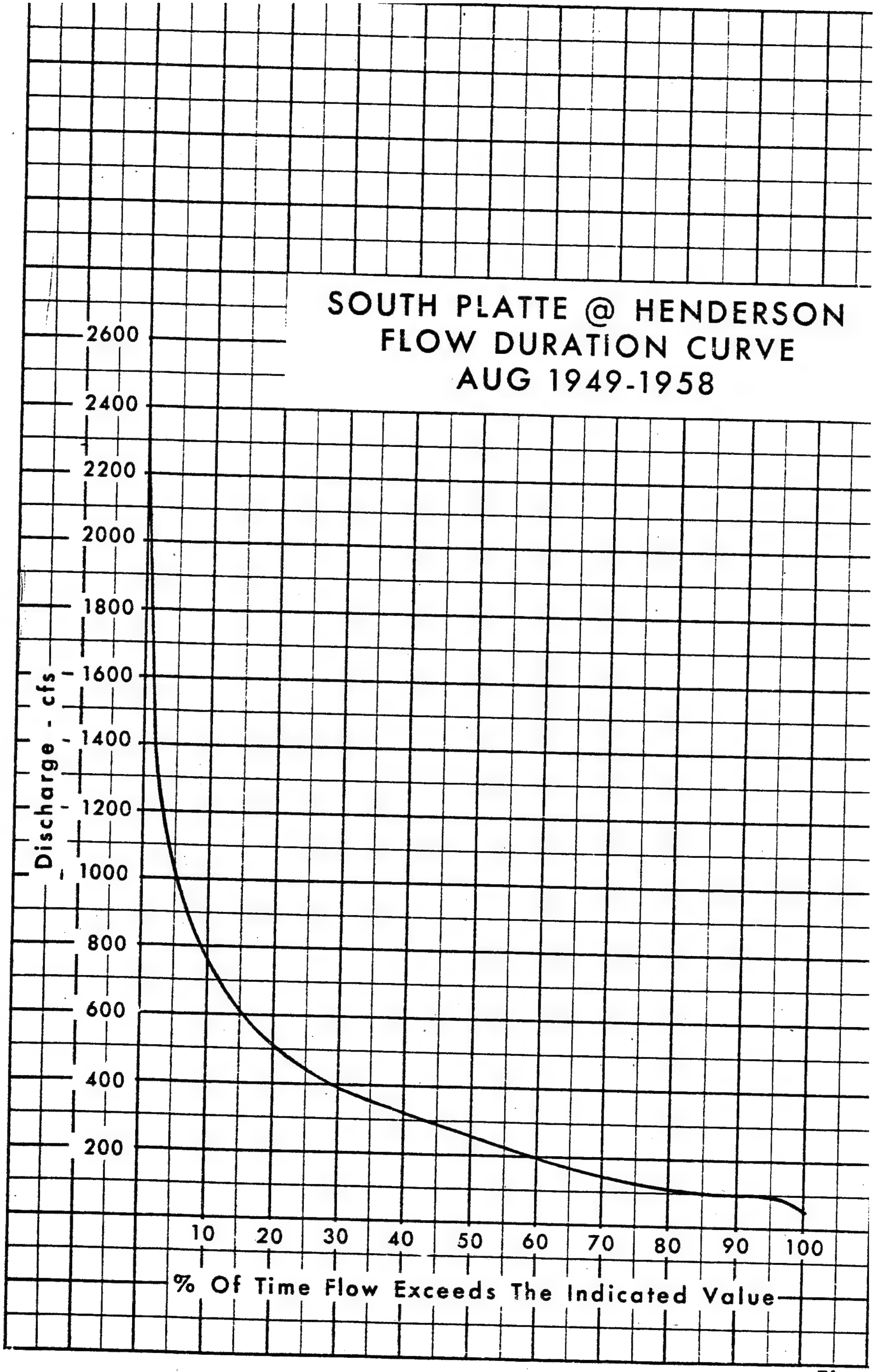

SOUTH PLATTE @ HENDERSON
FLOW DURATION CURVE
JUN 1949-1958
Discharge - cfs
6800
6400
6000
5600
5200
4800
4400
4000
3600
3200
2800
2400
2000
1600
1200
800
400
10
20
30
40
50
60
70
80
90
100
% Of Time Discharge Exceeds The Indicated Value

Fi.

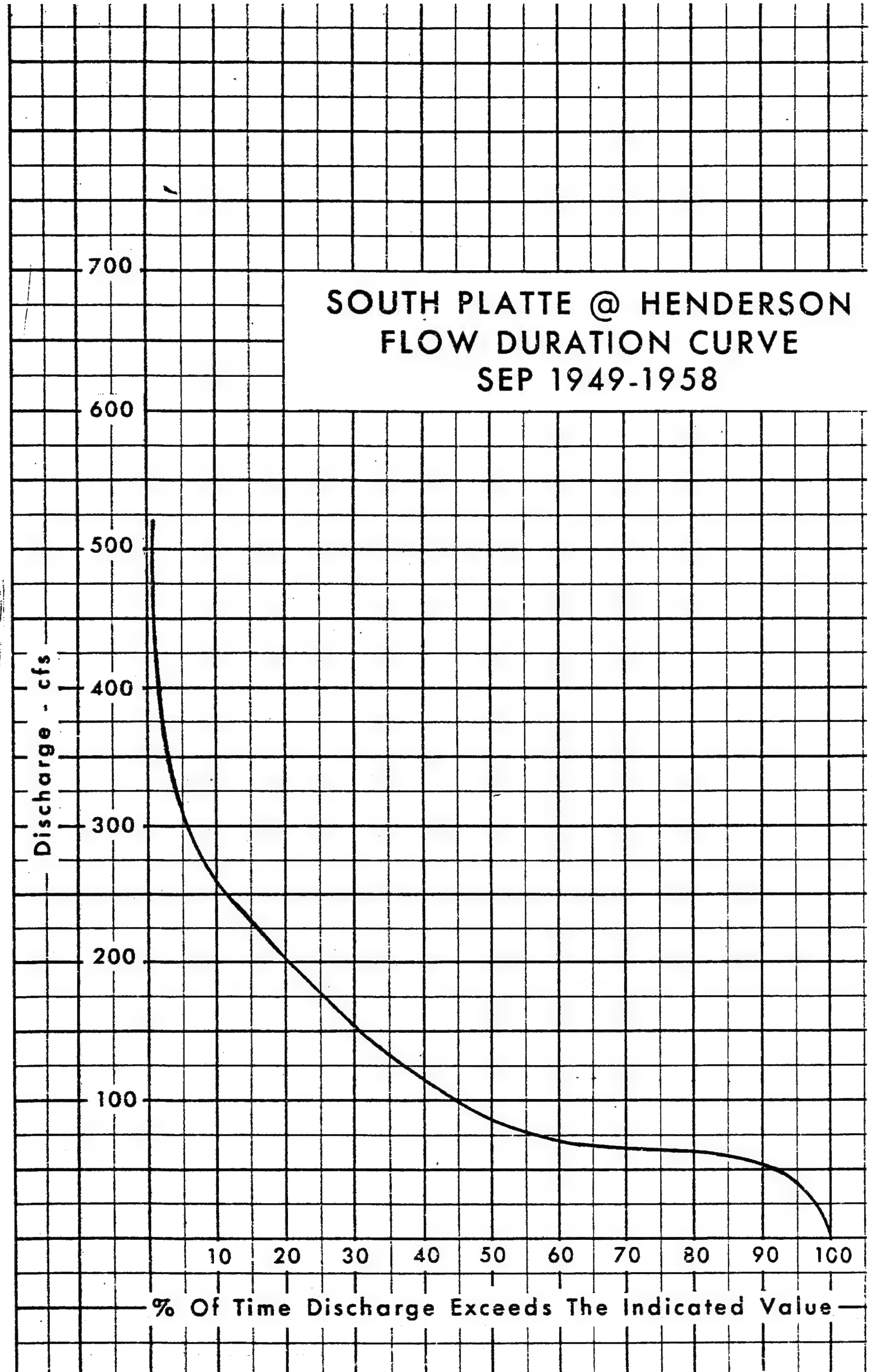


SOUTH PLATTE @ HENDERSON
FLOW DURATION CURVE
JUL 1949-1958
Discharge - cfs
2600
2400
2200
2000
1800
1600
1400
1200
1000
800
600
400
200
10
20
30
40
50
60
70
80
90
100
% Of Time Discharge Exceeds The Indicated Value

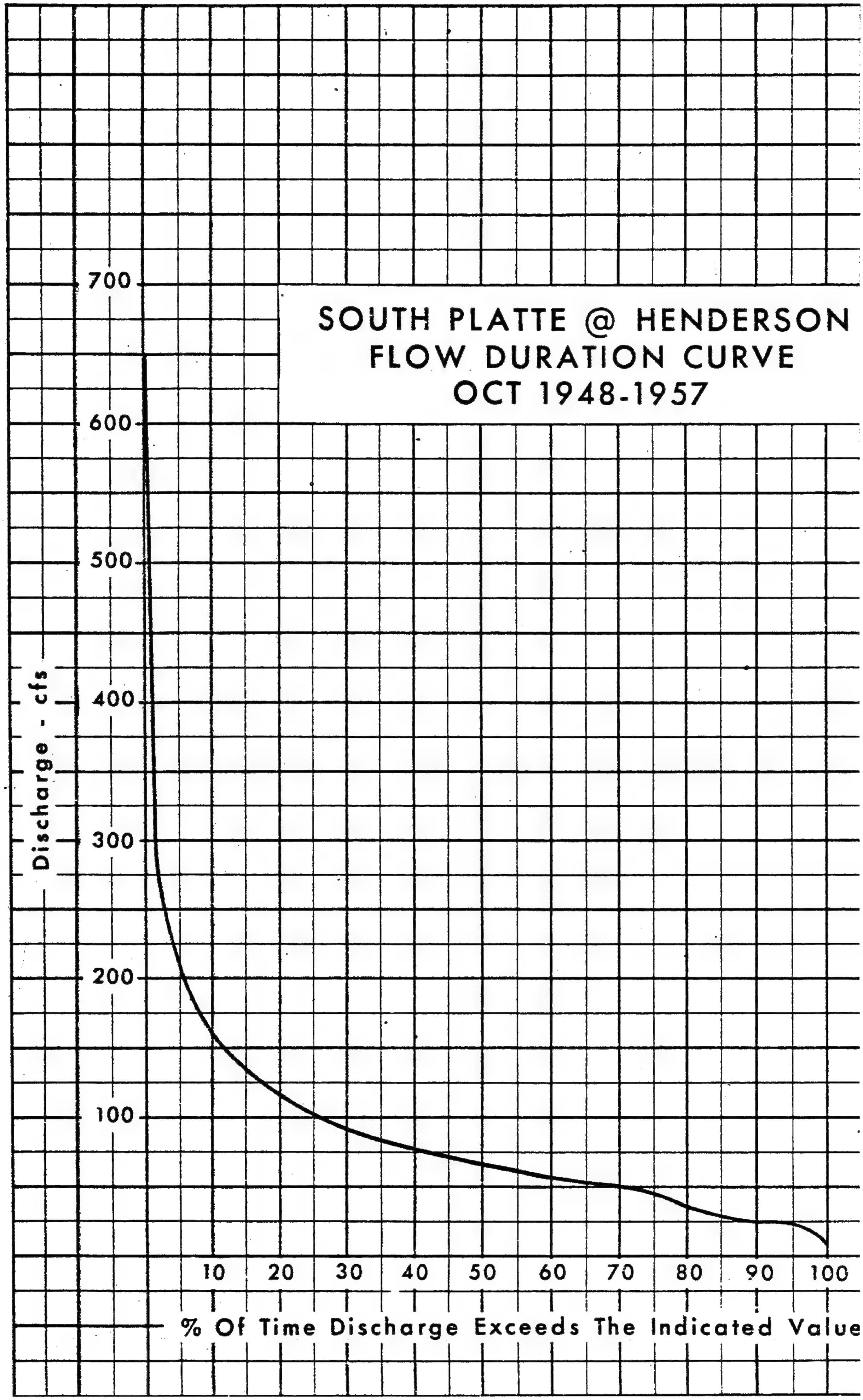
Fig



SOUTH PLATTE @ HENDERSON
FLOW DURATION CURVE
AUG 1949-1958
Discharge - cfs
2600
2400
2200
2000
1800
1600
1400
1200
1000
800
600
400
200
10
20
30
40
50
60
70
80
90
100
% Of Time Flow Exceeds The Indicated Value


SOUTH PLATTE @ HENDERSON
FLOW DURATION CURVE
SEP 1949-1958
Discharge - cfs
700
600
500
400
300
200
100
10
20
30
40
50
60
70
80
90
100
% Of Time Discharge Exceeds The Indicated Value

Fi


SOUTH PLATTE @ HENDERSON
FLOW DURATION CURVE
OCT 1948-1957
Discharge - cfs
700
600
500
400
300
200
100
10
20
30
40
50
60
70
80
90
100
% Of Time Discharge Exceeds The Indicated Value

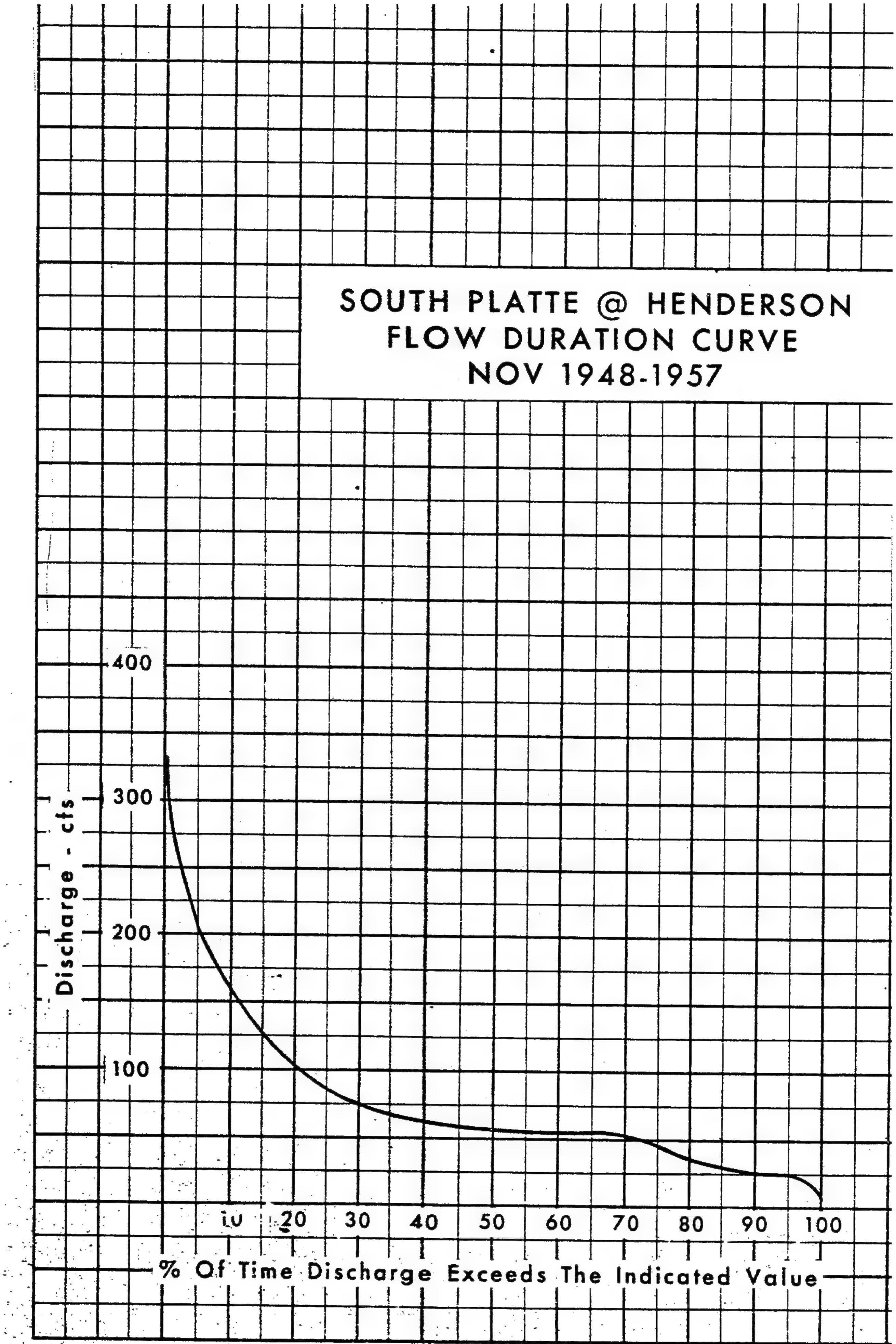
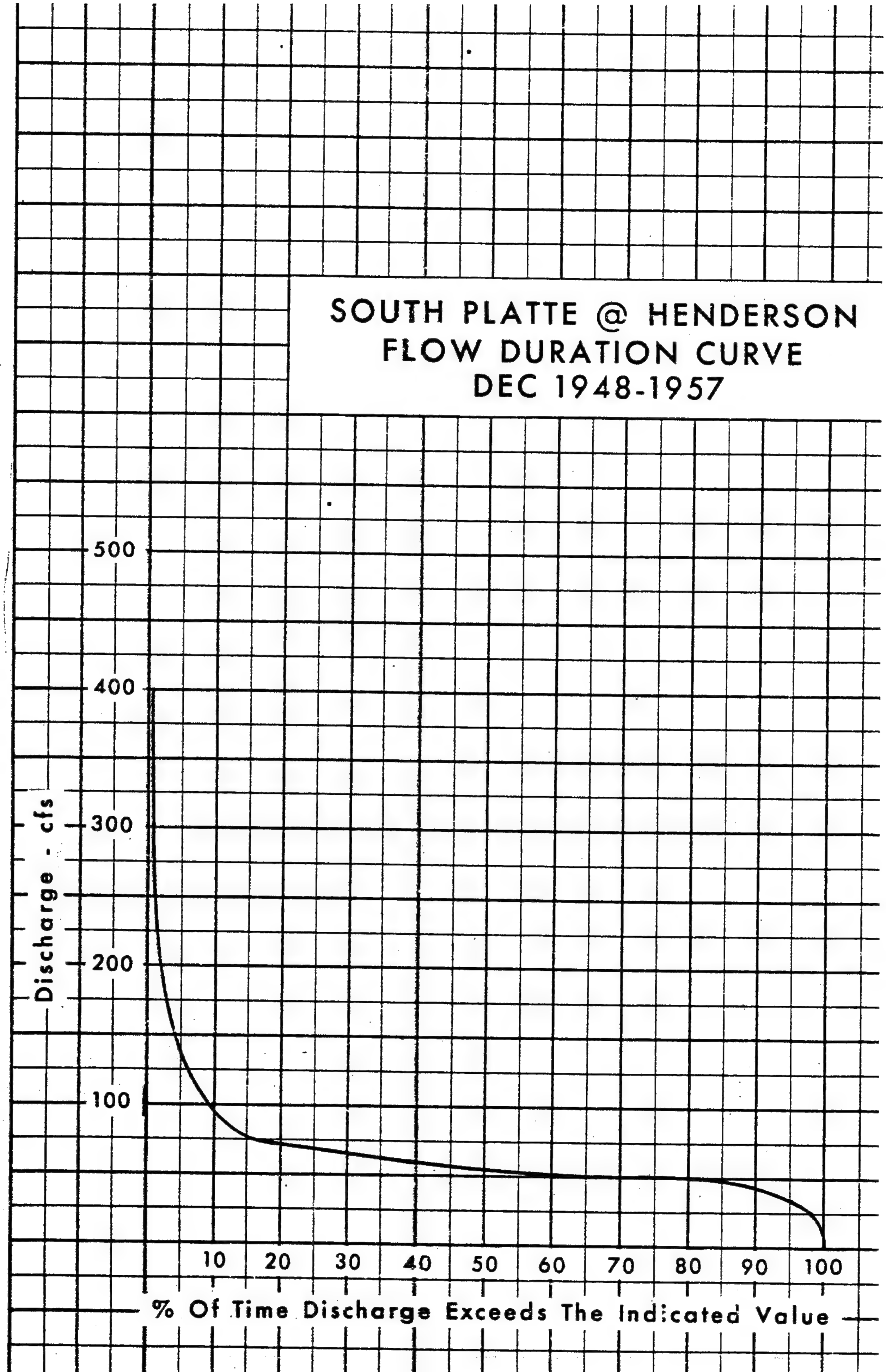
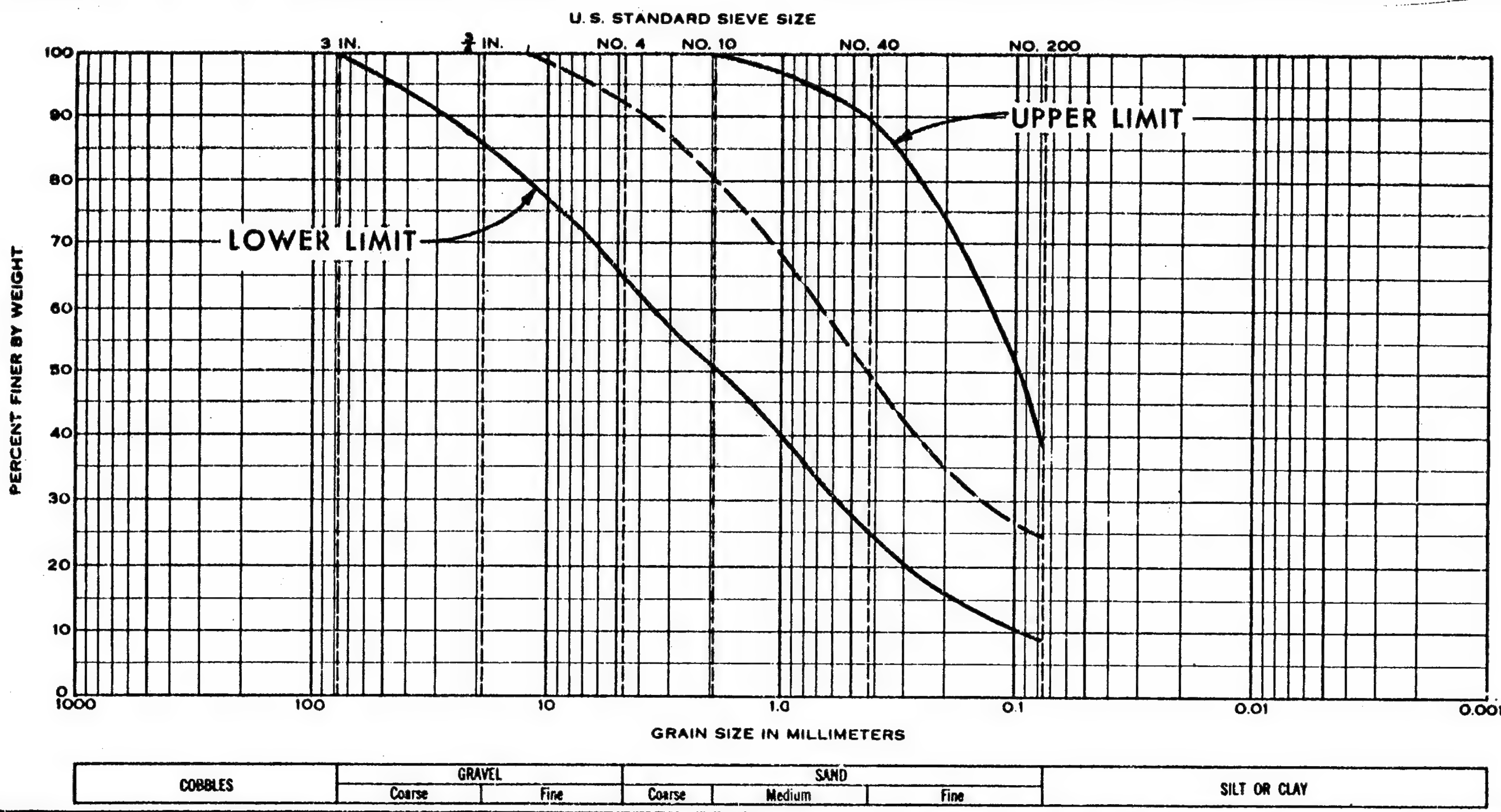

SOUTH PLATTE @ HENDERSON
FLOW DURATION CURVE
NOV 1948-1957
Discharge - cfs
400
300
200
100
10
20
30
40
50
60
70
80
90
100
% Of Time Discharge Exceeds The Indicated Value

Fig.


SOUTH PLATTE @ HENDERSON
FLOW DURATION CURVE
DEC 1948-1957
Discharge - cfs
500
400
300
200
100
10
20
30
40
50
60
70
80
90
100
% Of Time Discharge Exceeds The Indicated Value

Fig



Sample No.	Elev or Depth	Classification	NatWC	LL	PL	PI

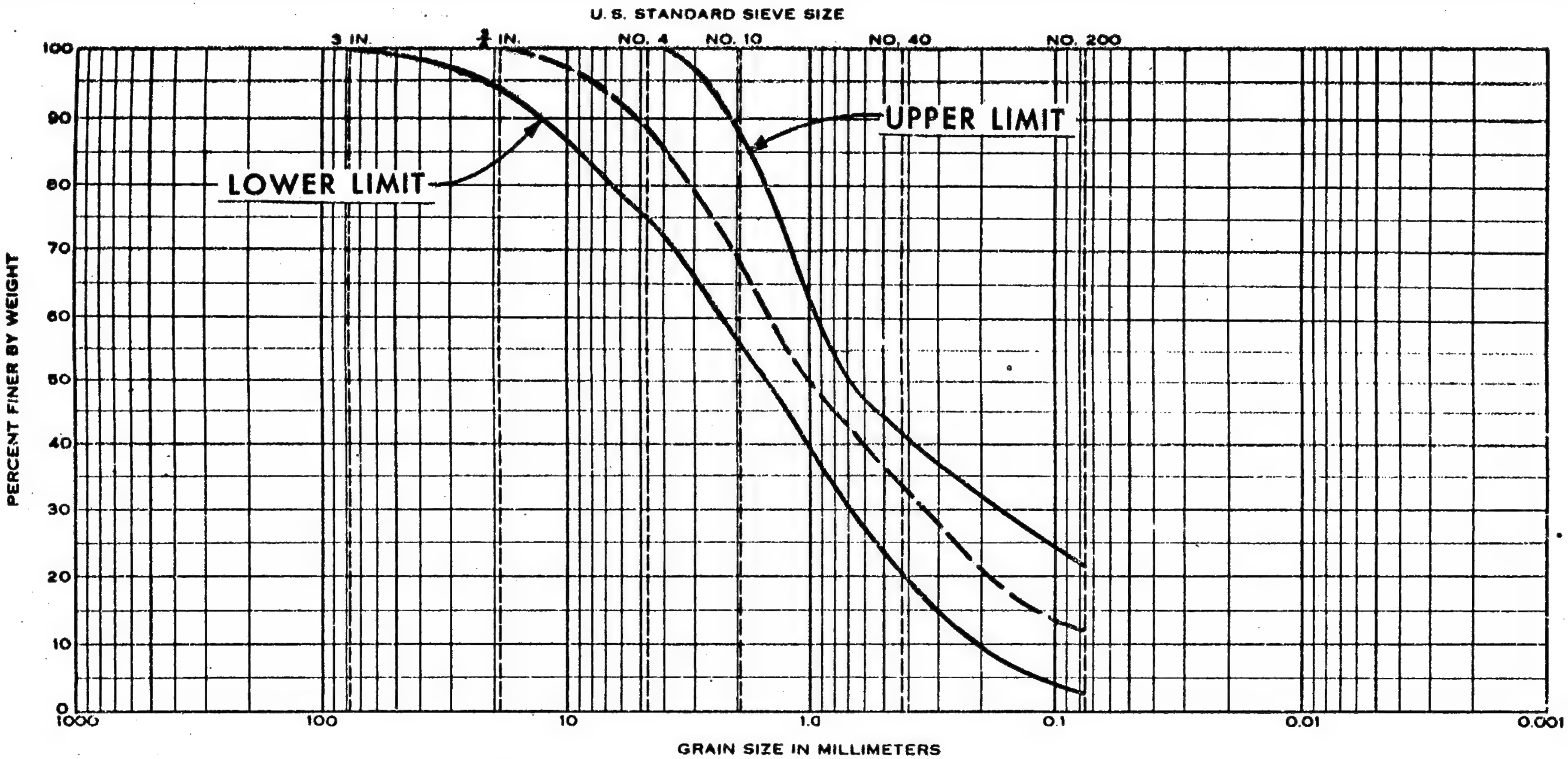
TYPICAL GRADATION OF SATURATED FOUNDATION SOILS

Project PROGRAM FOR RECLAMATION OF SURFACE AQUIFER

Area ON ARSENAL

Boring No.

PLATE



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Elev or Depth	Classification	NatWC	LL	PL	PI
TYPICAL GRADATION OF SATURATED FOUNDATION SOILS						

Project PROGRAM FOR RECLAMATION OF SURFACE AQUIFER

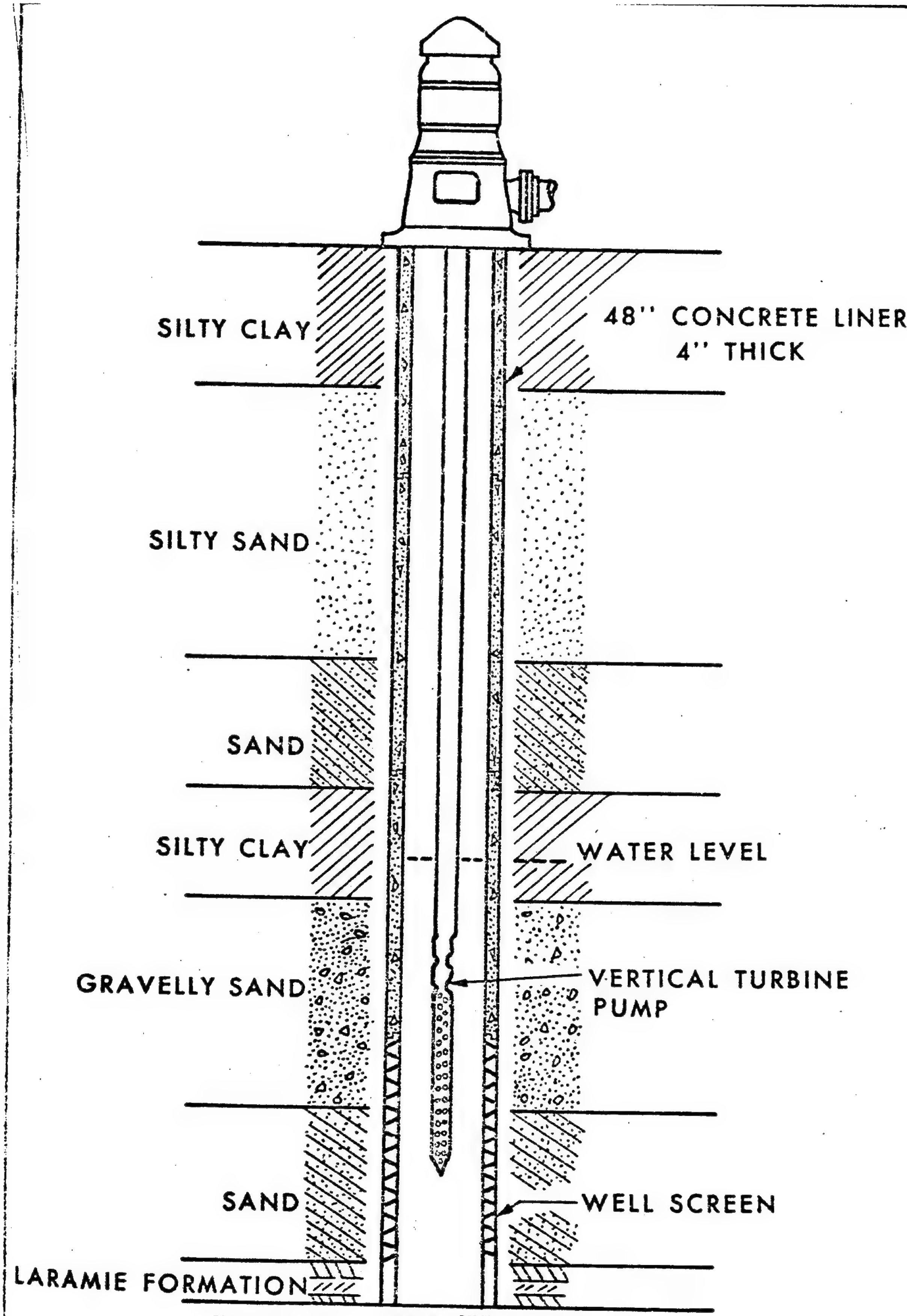
Area OFF ARSENAL

Boring No.

Date

GRADATION CURVES

PLATE 11



TYPICAL IRRIGATION WELL

ROCKY MOUNTAIN ARSENA
PROGRAM FOR RECLAMAT
OF SURFACE AQUIFER

VOLUMN OF CONTAMINATED WATER

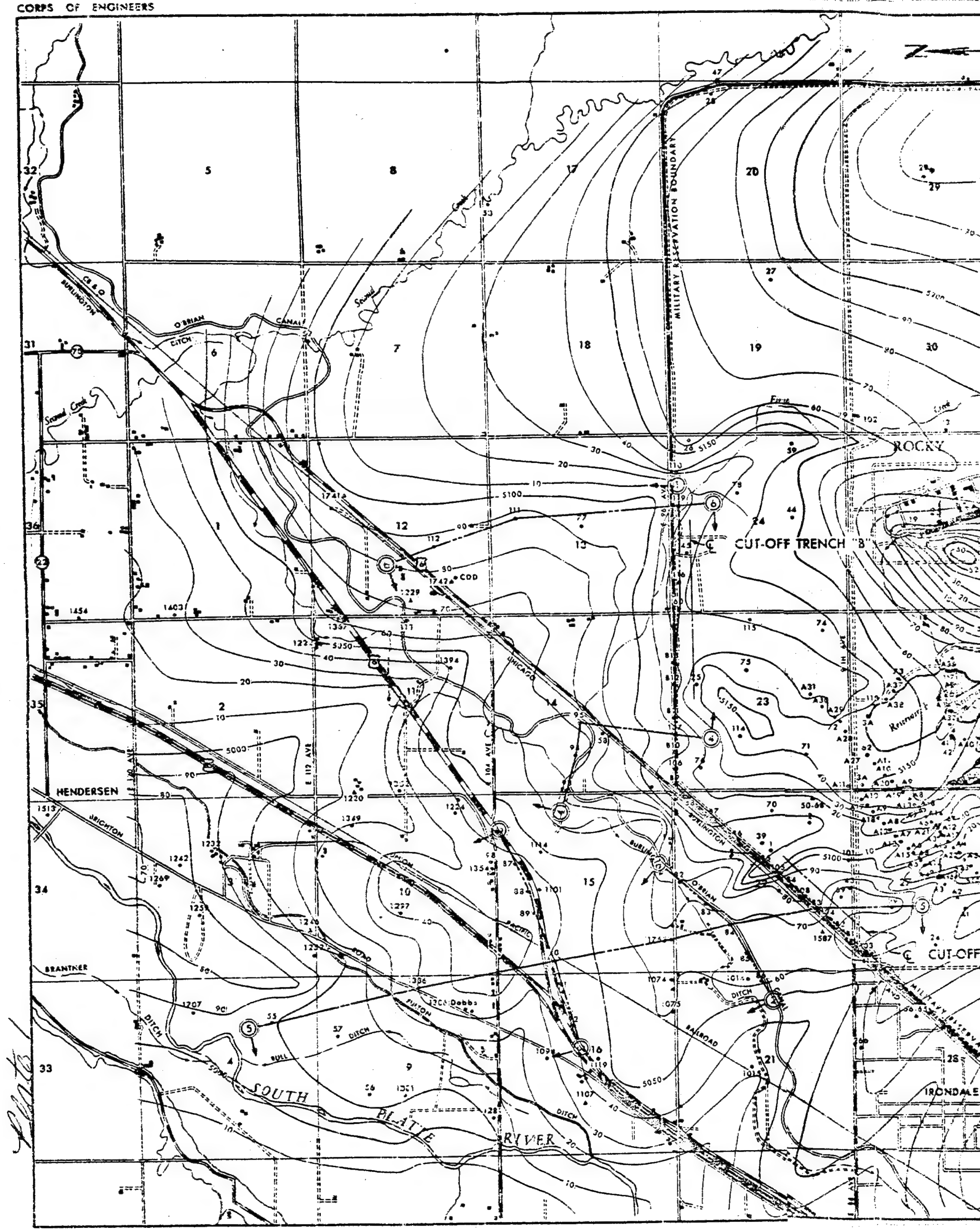
DEGREE OF CONTAM IN PPM CHLORIDE	OFF ARSENAL				ON ARSENAL		TOTALS	
	AREA A		AREA B					
	CU. FT.	GALLONS	CU. FT.	GALLONS	CU. FT.	GALLONS	CU. FT.	GALLONS
200-500	149,720,000	1,120,055,000	142,154,000	1,063,454,000	54,259,000	405,912,000	346,133,000	2,589,421,000
500-1000	84,658,000	633,326,000	91,207,000	682,320,000	78,898,000	590,236,000	254,763,000	1,905,882,000
1000-2000	66,218,000	495,377,000	9,453,000	70,718,000	150,426,000	1,125,337,000	226,097,000	1,691,432,000
2000-3000	-	-	2,352,000	17,595,000	111,019,000	830,533,000	113,371,000	848,128,000
3000-4000	-	-	-	-	36,133,000	270,311,000	36,133,000	270,311,000
4000-5000	-	-	-	-	4,077,000	30,500,000	4,077,000	30,500,000
5000+	-	-	-	-	1,032,000	7,720,000	1,032,000	7,720,000
TOTALS	300,596,000	2,248,758,000	245,166,000	1,834,087,000	435,844,000	3,260,549,000	981,606,000	7,343,394,000

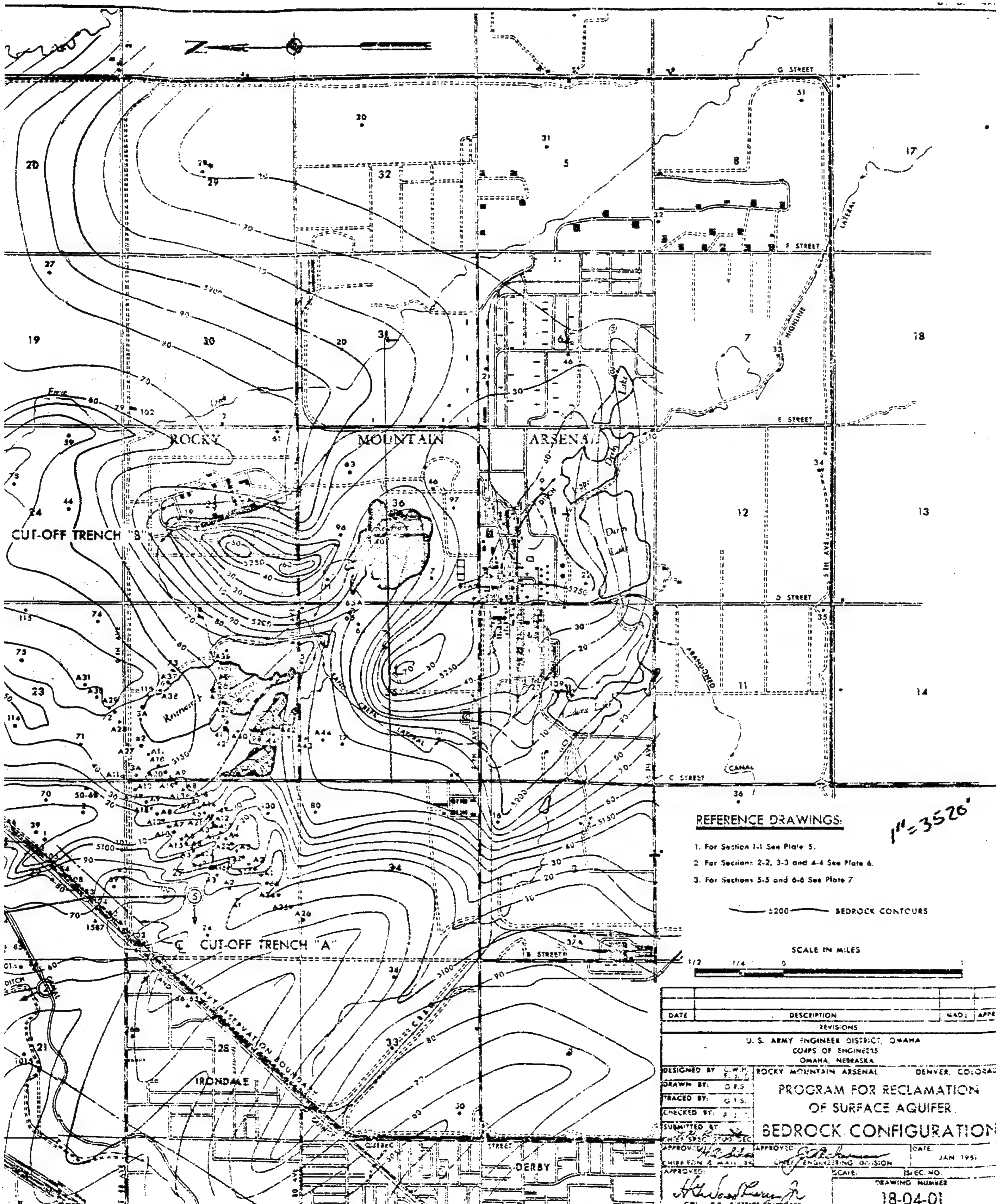
ROCKY MOUNTAIN ARSENAL

PROGRAM FOR RECLAMATION OF SURFACE AQUIFER

TABLE

CORPS OF ENGINEERS

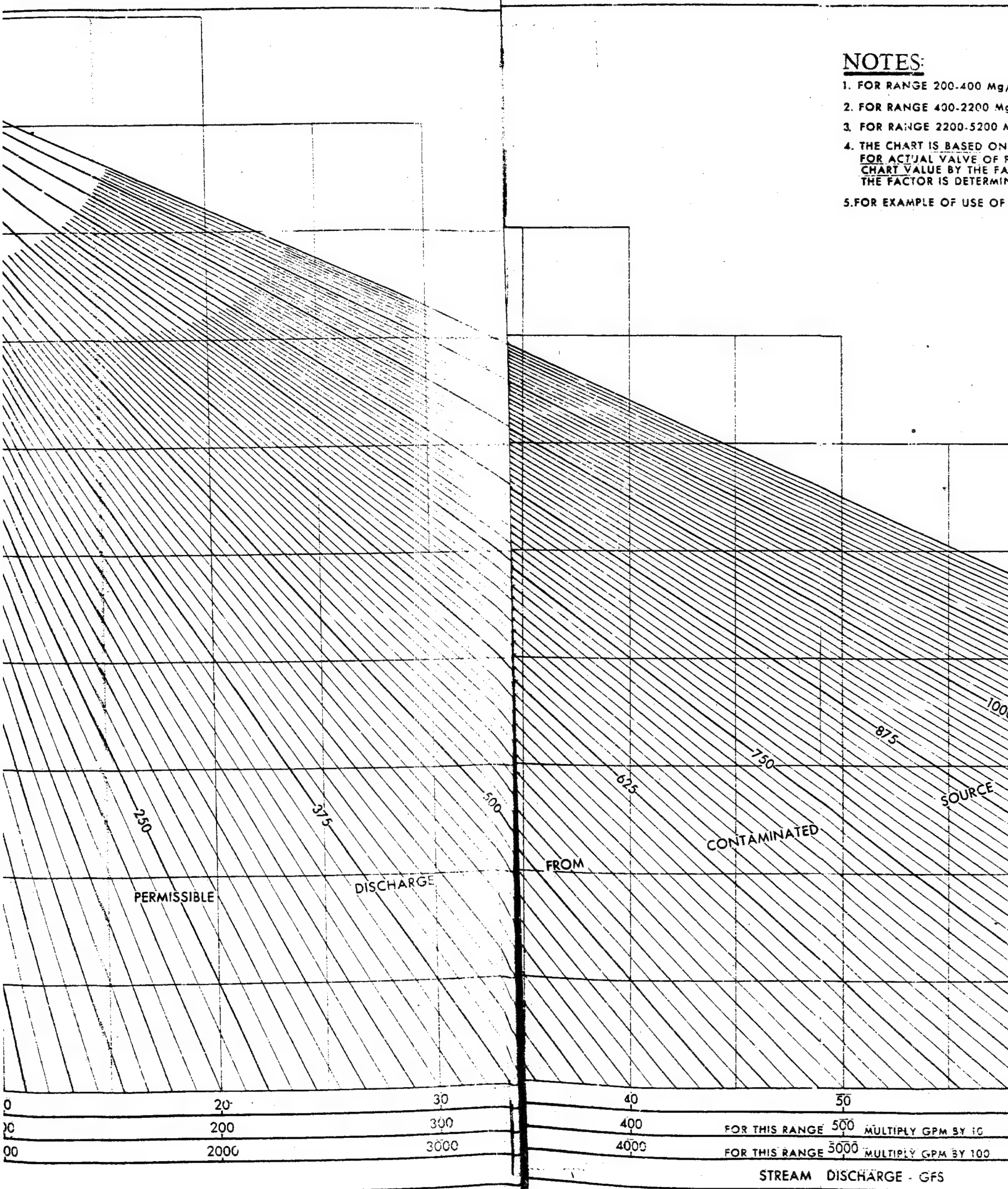




G STREET
F STREET
E STREET
D STREET
C STREET
B STREET
ROCKY MOUNTAIN ARSENAL
CUT-OFF TRENCH "B"
CUT-OFF TRENCH "A"
HIGHLINE
LATERAL
ABANDONED
CANAL
MILITARY RESERVATION BOUNDARY
IRONDALE
DERBY
1"=3520'
REFERENCE DRAWINGS:
1. For Section 1-1 See Plate 5.
2. For Sections 2-2, 3-3 and 4-4 See Plate 6.
3. For Sections 5-5 and 6-6 See Plate 7.
5200 BEDROCK CONTOURS
SCALE IN MILES
1/2 1/4 0 1
DATE
DESCRIPTION
MADE
APPR
REVISIONS
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA
DESIGNED BY
DRAWN BY: G.R.S.
TRACED BY: G.T.S.
CHECKED BY:
SUBMITTED BY:
CHIEF SPEC. STUD. SEC.
APPROVED:
CHIEF FDN. & MAT. SEC.
APPROVED:
CHIEF, ENGINEERING DIVISION
APPROVED:
COL., C.E. DISTRICT ENGINEER
ROCKY MOUNTAIN ARSENAL
DENVER, COLORADO
PROGRAM FOR RECLAMATION
OF SURFACE AQUIFER
BEDROCK CONFIGURATION
DATE: JAN 1955
SCALE:
SPEC. NO.
DRAWING NUMBER
18-04-01

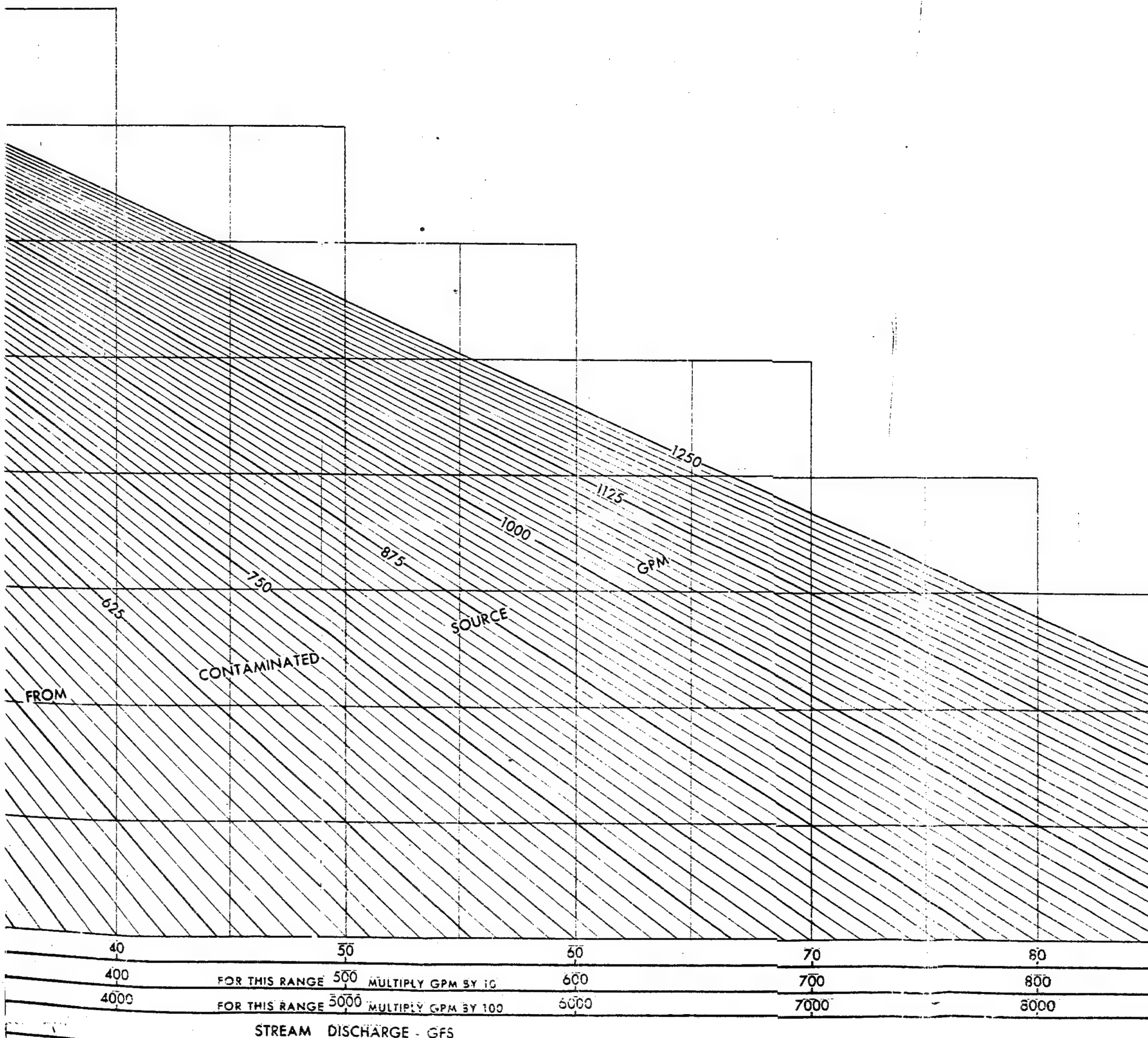
NOTES:

1. FOR RANGE 200-400 Mg/
2. FOR RANGE 400-2200 Mg
3. FOR RANGE 2200-5200 M
4. THE CHART IS BASED ON
FOR ACTUAL VALVE OF F
CHART VALUE BY THE FA
THE FACTOR IS DETERMIN
5. FOR EXAMPLE OF USE OF



NOTES:

1. FOR RANGE 200-400 Mg/L CHLORIDES IN CONTAMINATED SOURCE USE CHART 1
2. FOR RANGE 400-2200 Mg/L CHLORIDES IN CONTAMINATED SOURCE USE CHART 2
3. FOR RANGE 2200-5200 Mg/L CHLORIDES IN CONTAMINATED SOURCE USE CHART 3
4. THE CHART IS BASED ON ZERO CONCENTRATION OF CHLORIDE IN THE RECEIVING STREAM. FOR ACTUAL VALVE OF PERMISSIBLE DISCHARGE FROM CONTAMINATED SOURCE IN GPM MULTIPLY THE CHART VALUE BY THE FACTOR FOR THE MONTH AND STREAM SHOWN IN THE TABLE ON THIS CHART. THE FACTOR IS DETERMINED FROM THE AVERAGE CONCENTRATION OF CHLORIDE DURING EACH MONTH.
5. FOR EXAMPLE OF USE OF CHART, SEE CHART 1, Fig. IIa



U. S

CONTAMINATED SOURCE USE CHART 1
CONTAMINATED SOURCE USE CHART 2
N CONTAMINATED SOURCE USE CHART 1
ATION OF CHLORIDE IN THE RECEIVING STREAM.
IARGE FROM CONTAMINATED SOURCE IN GPM MULTIPLY THE
ONTH AND STREAM SHOWN IN THE TABLE ON THIS CHART.
ERAGE CONCENTRATION OF CHLORIDE DURING EACH MONTH.

T 1, Fig. IIa

FACTOR		
MONTH	SOUTH PLATTE RIVER	IRRIGATI(DITCHE:
JAN.	0.62	0.30
FEB.	0.64	0.17
MAR.	0.59	0.16
APR.	0.48	0.34
MAY	0.44	0.39
JUNE	0.53	0.43
JULY	0.60	0.48
AUG.	0.68	0.52
SEPT.	0.63	0.57
OCT.	0.54	0.39
NOV.	0.66	0.45
DEC.	0.60	0.35

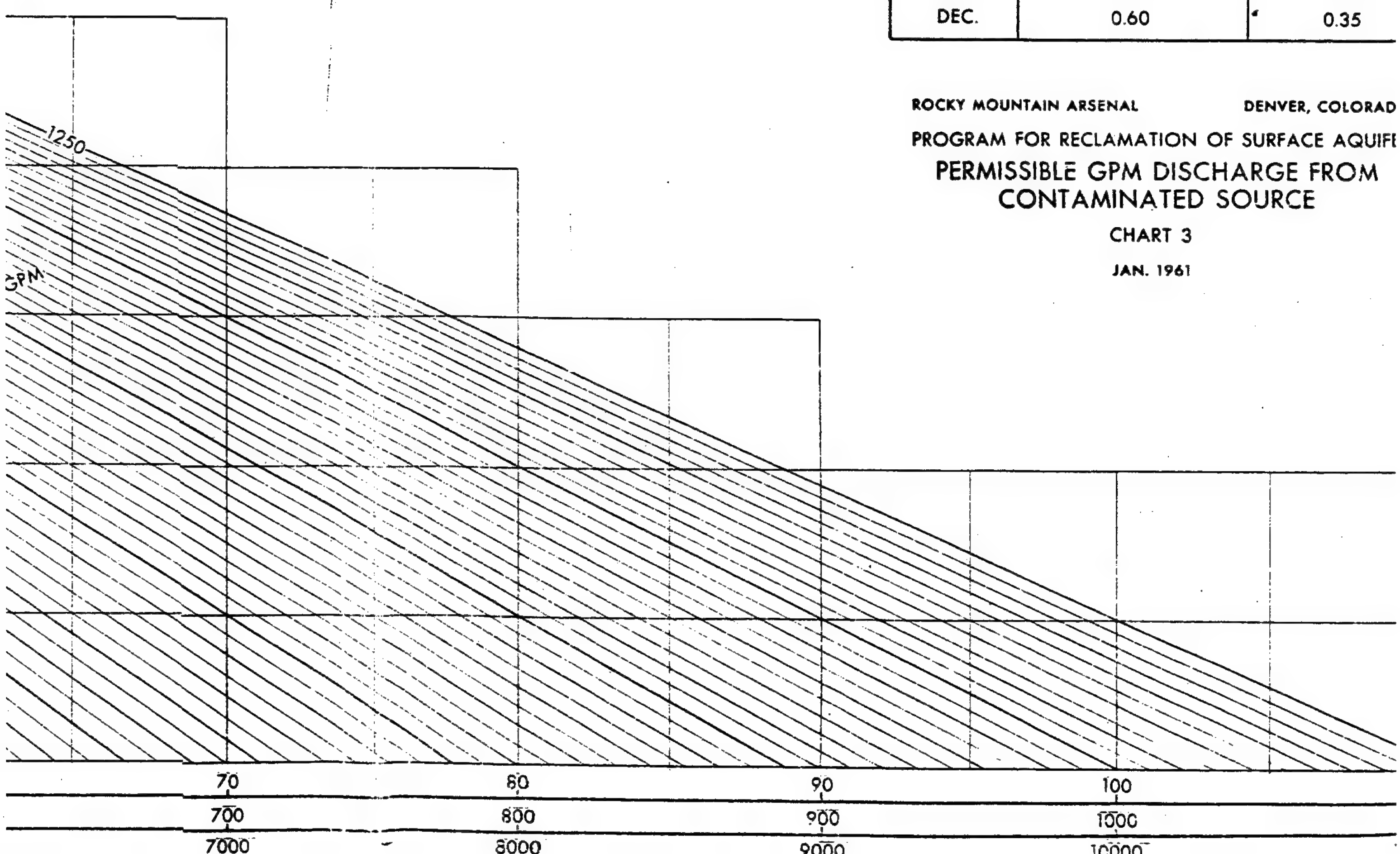
ROCKY MOUNTAIN ARSENAL DENVER, COLORAD

PROGRAM FOR RECLAMATION OF SURFACE AQUIFI

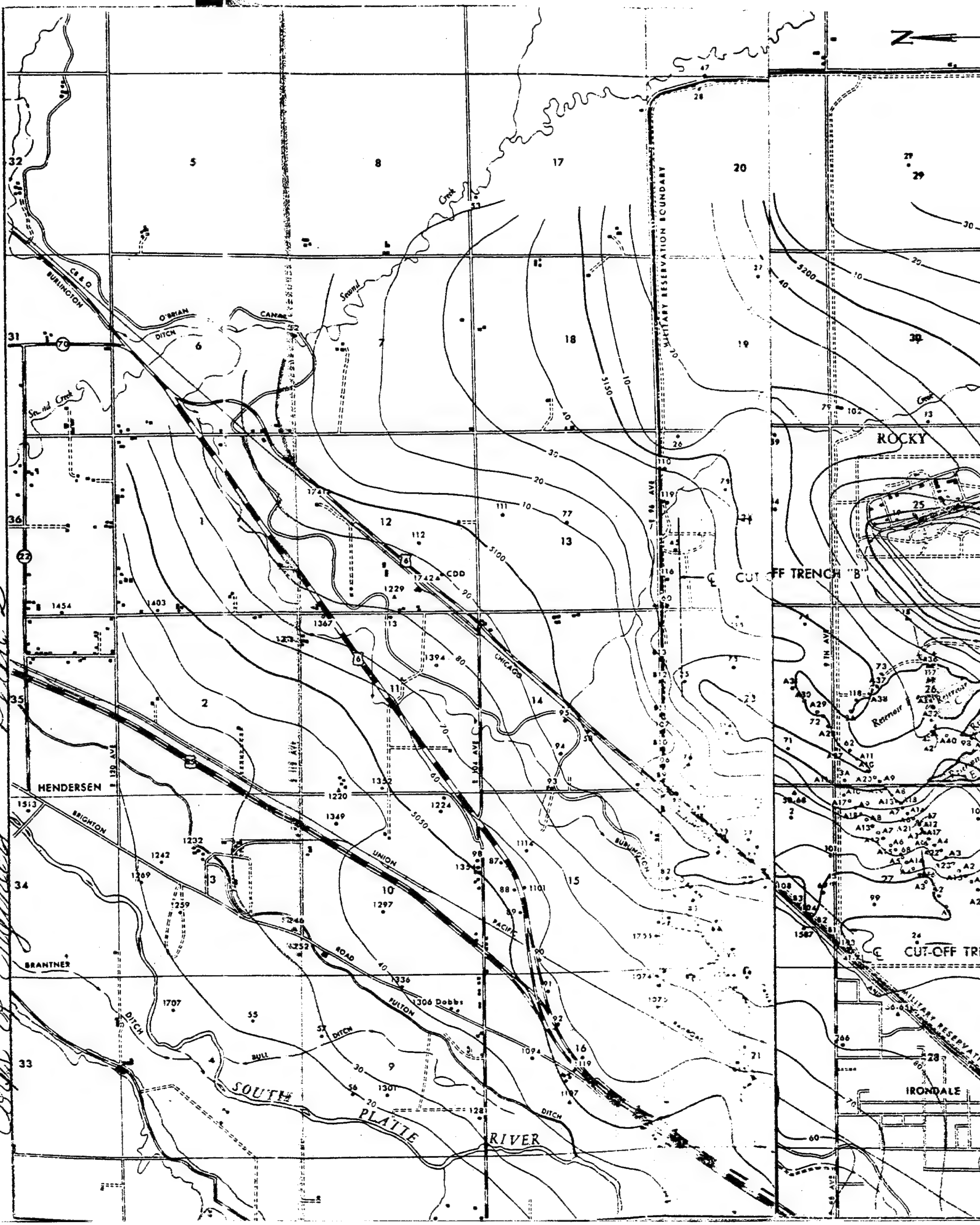
PERMISSIBLE GPM DISCHARGE FROM CONTAMINATED SOURCE

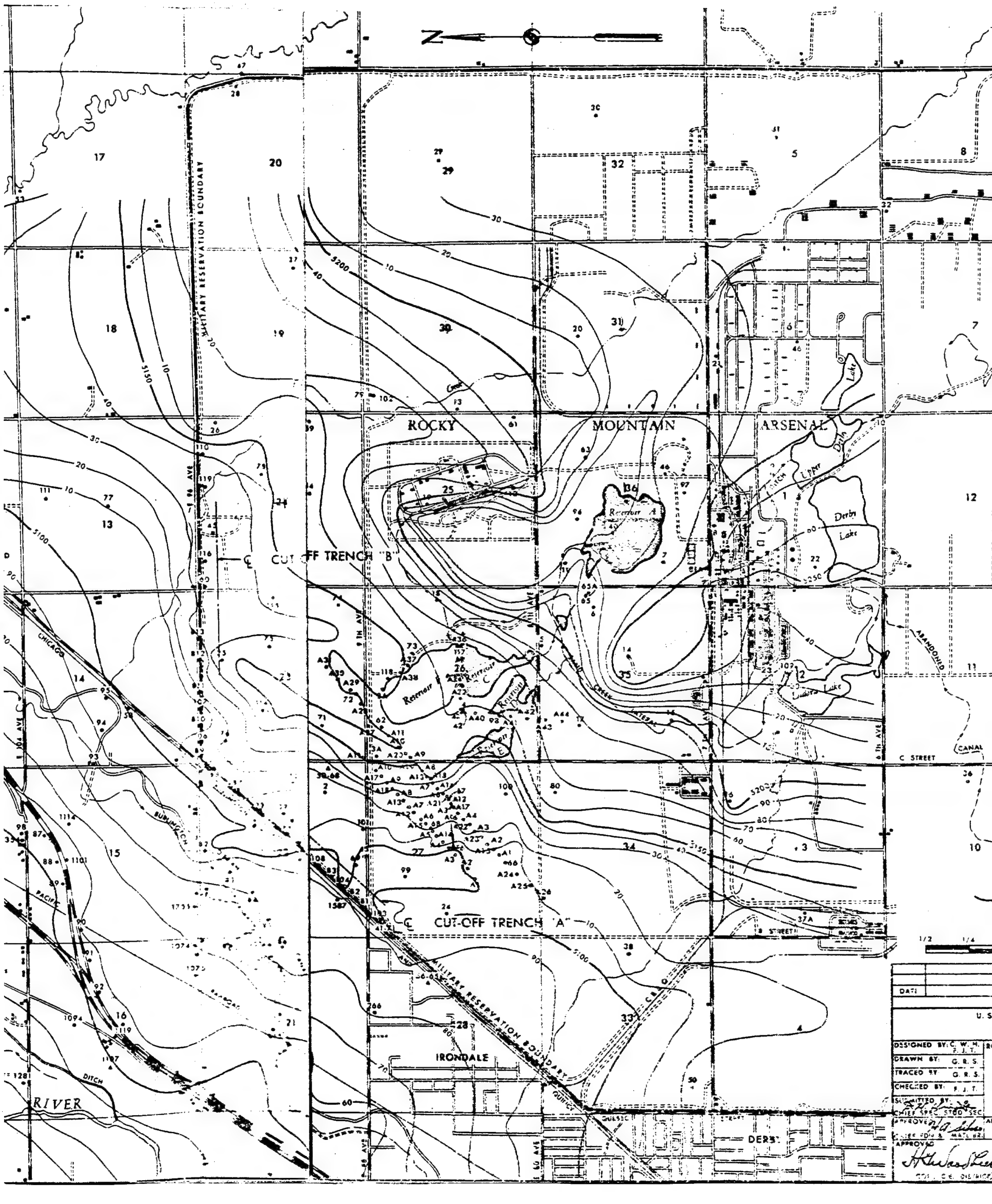
CHART 3

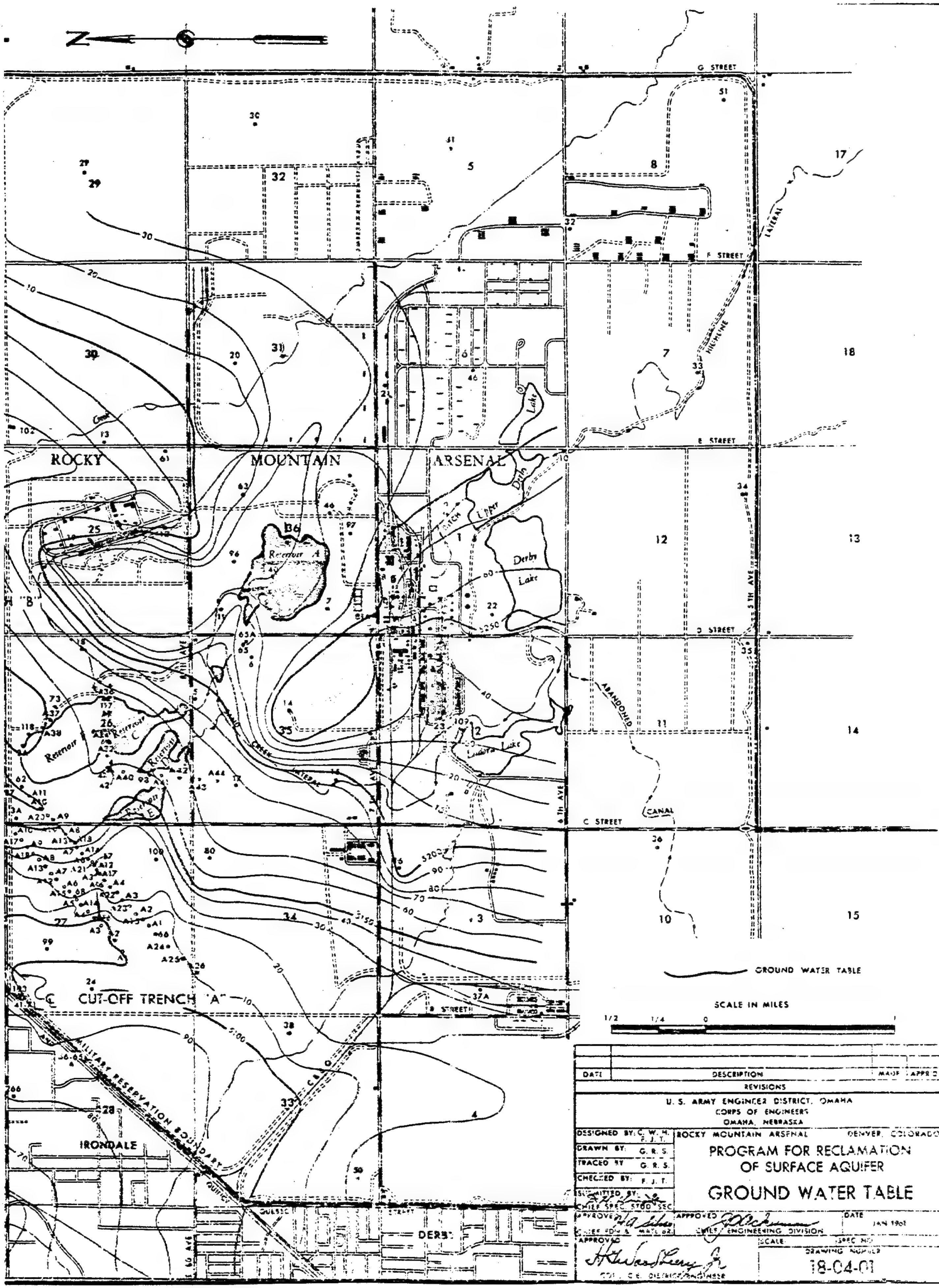
JAN. 1961

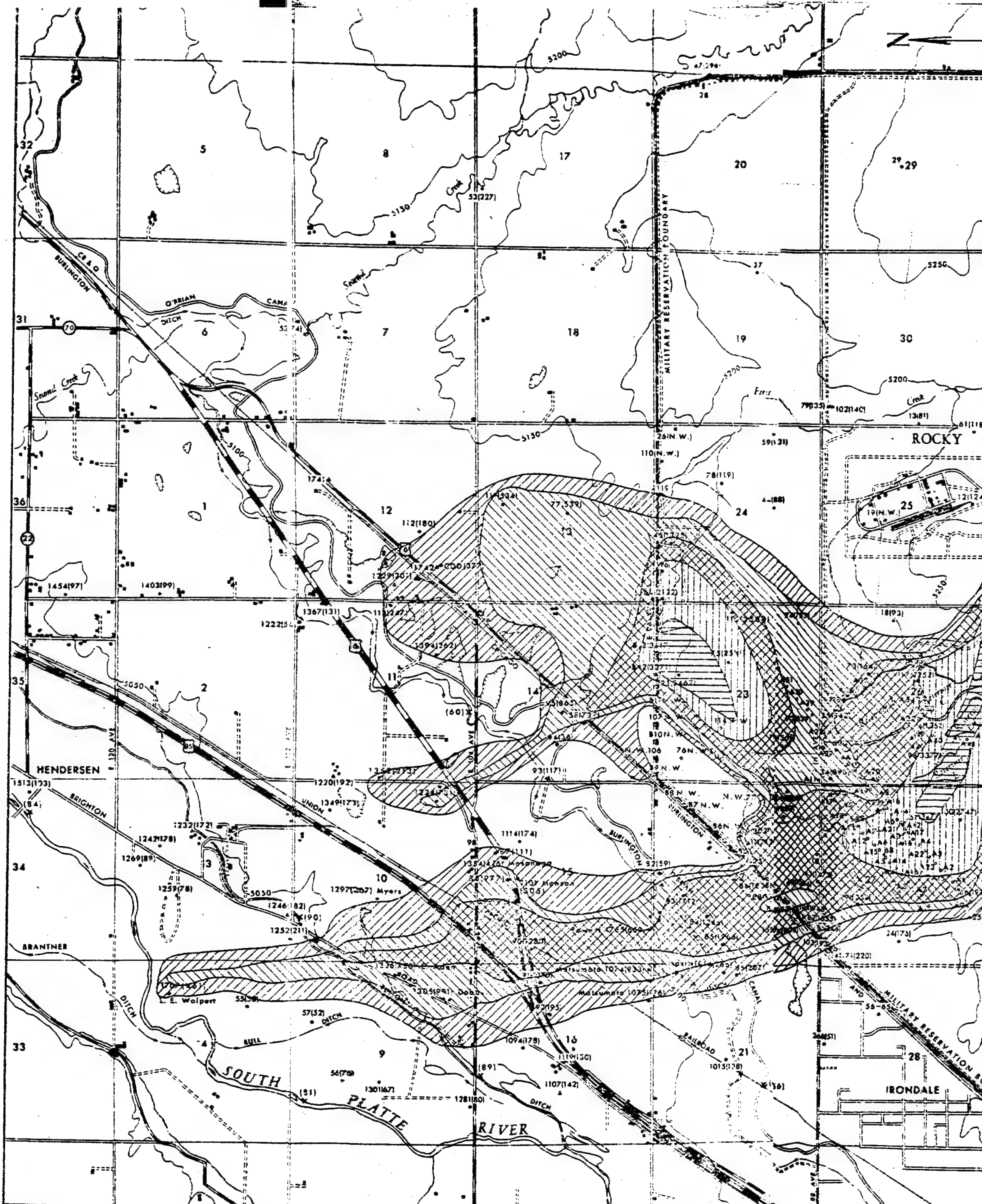


Corps of Engineers Plate 2




ROCKY MOUNTAIN ARSENAL
CUT-OFF TRENCH "B"
CUT-OFF TRENCH "A"
MILITARY RESERVATION BOUNDARY
Reservoir A
Reservoir C
Reservoir F
Reservoir E
Derby Lake
Upper Derby Lake
Ladora Lake
IRONDALE
DERBY
RIVER
CHICAGO
BURLINGTON
PACIFIC
DITCH
C STREET
B STREET
ABANDONED
CANAL
DESIGNED BY: C. W. M. F. J. T.
DRAWN BY: G. R. S.
TRACED BY: G. R. S.
CHECKED BY: F. J. T.
DATE
U. S.


N
G STREET
F STREET
E STREET
D STREET
C STREET
B STREET
ROCKY MOUNTAIN ARSENAL
Reservoir A
Derby Lake
Upper Derby
Lake Ladora
CUT-OFF TRENCH "A"
MILITARY RESERVATION BOUNDARY
IRONDALE
DERBY
ABANDONED
CANAL
HIGHLINE
LATERAL
5TH AVE
6TH AVE
7TH AVE
GROUND WATER TABLE
SCALE IN MILES
1/2
1/4
0
1
DATE
DESCRIPTION
MADE
APPR'D
REVISIONS
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA
DESIGNED BY: C. W. H. F. J. T.
DRAWN BY: G. R. S.
TRACED BY: G. R. S.
CHECKED BY: F. J. T.
SUBMITTED BY:
CHIEF SPEC. STUD. SEC.
APPROVED
CHIEF FDN & MAT'L BR.
APPROVED
CHIEF, ENGINEERING DIVISION
APPROVED
COL., C.E. DISTRICT ENGINEER
ROCKY MOUNTAIN ARSENAL
DENVER, COLORADO
PROGRAM FOR RECLAMATION
OF SURFACE AQUIFER
GROUND WATER TABLE
DATE
JAN 1961
SCALE
SPEC. NO.
DRAWING NUMBER
18-04-01



ROCKY
HENDERSEN
IRONDALE
SOUTH PLATTE RIVER
MILITARY RESERVATION BOUNDARY
BURLINGTON
CB & Q
O'BRIAN
DITCH
CANAL
BRIGHTON
UNION
BRANTNER
BULL
DITCH
RAILROAD
Second Creek
First
Creek
E. 120 AVE
L. E. Wolpert
Myers
Matsumoto
Dobb

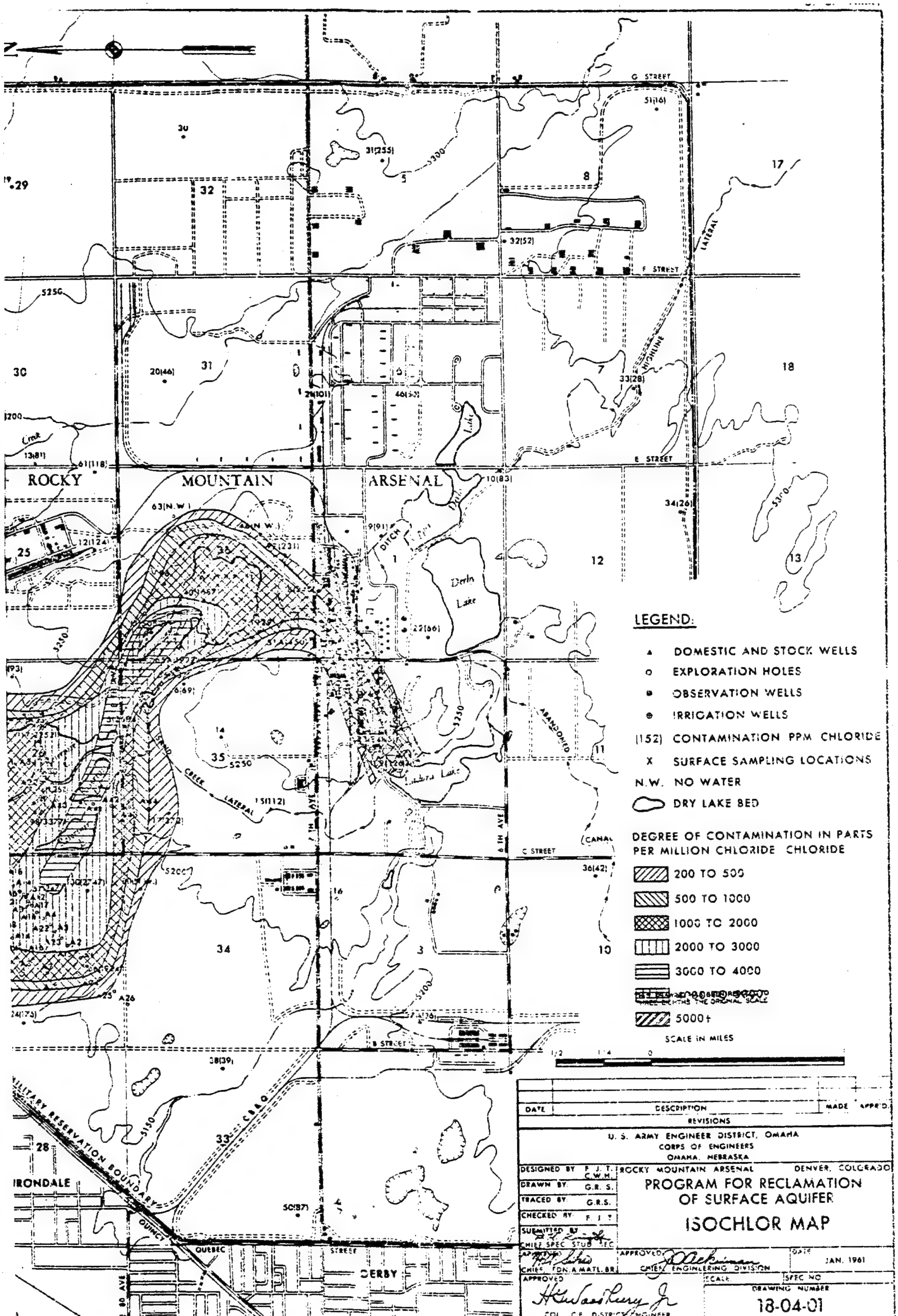
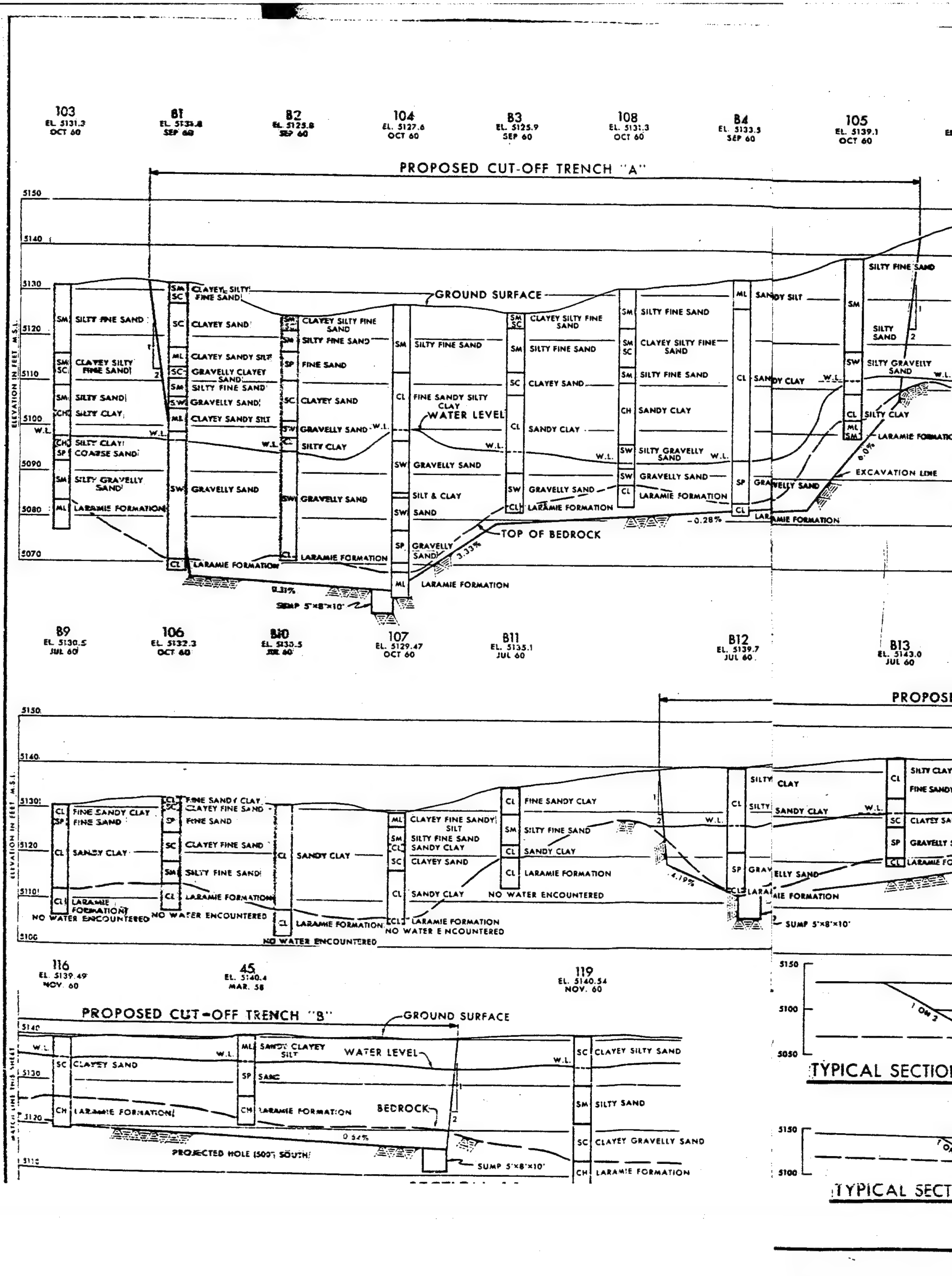
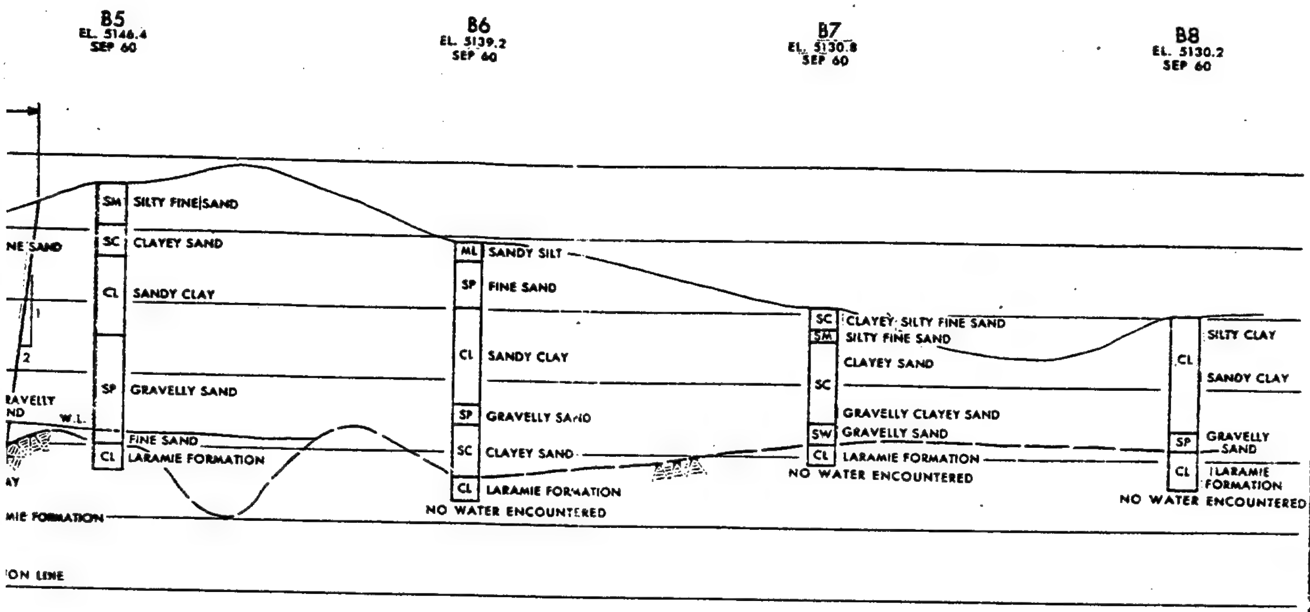
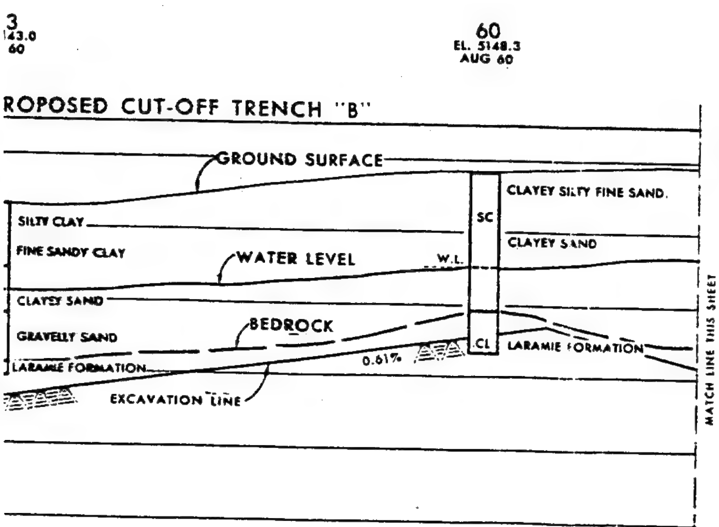

LEGEND:
DOMESTIC AND STOCK WELLS
EXPLORATION HOLES
OBSERVATION WELLS
IRRIGATION WELLS
(152) CONTAMINATION PPM CHLORIDE
X SURFACE SAMPLING LOCATIONS
N.W. NO WATER
DRY LAKE BED
DEGREE OF CONTAMINATION IN PARTS PER MILLION CHLORIDE CHLORIDE
200 TO 500
500 TO 1000
1000 TO 2000
2000 TO 3000
3000 TO 4000
5000+
SCALE IN MILES
ROCKY MOUNTAIN ARSENAL
G STREET
F STREET
E STREET
C STREET
B STREET
Derby Lake
Ladora Lake
HIGHLINE LATERAL
ABANDONED
DITCH
SAND CREEK LATERAL
MILITARY RESERVATION BOUNDARY
IRONDALE
QUEBEC
DERBY
DATE
DESCRIPTION
REVISIONS
MADE
APPR'D.
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA
DESIGNED BY F. J. T. C.W.H.
DRAWN BY G.R.S.
TRACED BY G.R.S.
CHECKED BY F. J. T.
SUBMITTED BY
CHIEF SPEC. STUD. SEC.
CHIEF FDN. & MATL. BR.
APPROVED
COL. C.E. DISTRICT ENGINEER
ROCKY MOUNTAIN ARSENAL
DENVER, COLORADO
PROGRAM FOR RECLAMATION OF SURFACE AQUIFER
ISOCHLOR MAP
CHIEF, ENGINEERING DIVISION
DATE JAN. 1961
SCALE
SPEC NO
DRAWING NUMBER
18-04-01

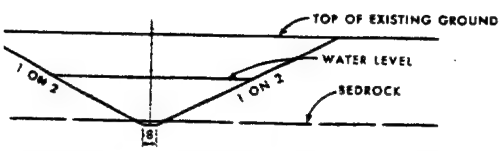
PLATE 4



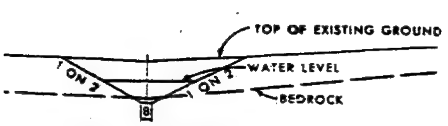
103 EL. 5131.3 OCT 60
B1 EL. 5133.8 SEP 60
B2 EL. 5125.8 SEP 60
104 EL. 5127.6 OCT 60
B3 EL. 5125.9 SEP 60
108 EL. 5131.3 OCT 60
B4 EL. 5133.5 SEP 60
105 EL. 5139.1 OCT 60
PROPOSED CUT-OFF TRENCH "A"
GROUND SURFACE
WATER LEVEL
TOP OF BEDROCK
EXCAVATION LINE
ELEVATION IN FEET M.S.L.
SILTY FINE SAND
CLAYEY SILTY FINE SAND
CLAYEY SAND
CLAYEY SANDY SILT
GRAVELLY CLAYEY SAND
SILTY SAND
SILTY CLAY
COARSE SAND
SILTY GRAVELLY SAND
GRAVELLY SAND
FINE SAND
FINE SANDY SILTY CLAY
SILT & CLAY
SANDY CLAY
LARAMIE FORMATION
SUMP 5'×8'×10'
W.L.
0.31%
3.33%
-0.28%
B9 EL. 5130.5 JUL 60
106 EL. 5132.3 OCT 60
B10 EL. 5130.5 JUL 60
107 EL. 5129.47 OCT 60
B11 EL. 5135.1 JUL 60
B12 EL. 5139.7 JUL 60
B13 EL. 5143.0 JUL 60
PROPOSE
FINE SANDY CLAY
CLAYEY FINE SANDY SILT
NO WATER ENCOUNTERED
4.19%
116 EL. 5139.49 NOV. 60
45 EL. 5140.4 MAR. 58
119 EL. 5140.54 NOV. 60
PROPOSED CUT-OFF TRENCH "B"
SANDY CLAYEY SILT
CLAYEY SILTY SAND
CLAYEY GRAVELLY SAND
BEDROCK
PROJECTED HOLE (500') SOUTH
0.54%
MATCH LINE THIS SHEET
TYPICAL SECTION
TYPICAL SECT







ECTION NORTHWEST BOUNDARY



. SECTION NORTH BOUNDARY

REFERENCE DRAWINGS:

1. For locations of sections see dwg. no. 18-04-01, Plate 1

THIS DRAWING HAS BEEN REDUCED TO THREE EIGHTHS THE ORIGINAL SCALE

REVISIONS

DATE	DESCRIPTION	MADE	APPR'D

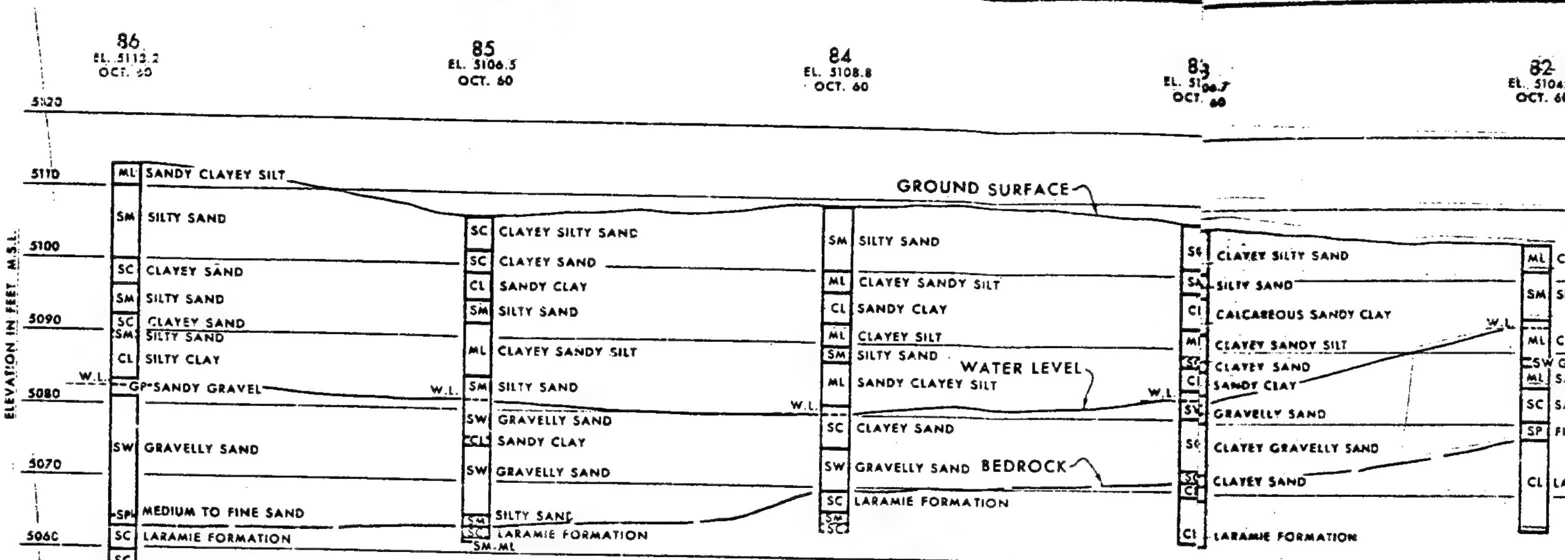
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

ROCKY MOUNTAIN ARSENAL — DENVER, COLORADO

PROGRAM FOR RECLAMATION
OF SURFACE AQUIFER
SECTIONS

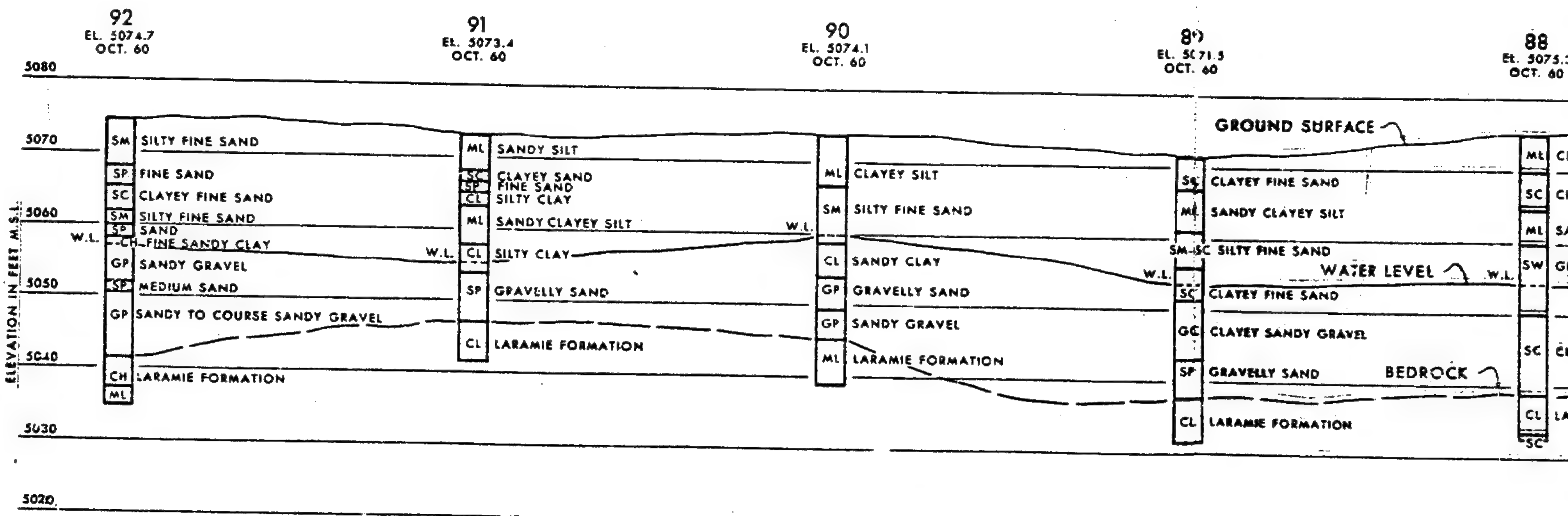
DESIGNED BY: C W N / F J T
DRAWN BY: G. L. S.
TRACED BY: C R S
CHECKED BY: C W N / F J T
SUBMITTED BY: S. K. K[illegible]
APPROVED: H. G. Sikes — CHIEF, FDN & MAT'L BRANCH
APPROVED: J. Ackerman — CHIEF, ENGINEERING DIVISION
APPROVED: H. Gwood Lucy, Jr. — COL., C. E., DISTRICT ENGINEER
DATE: JAN. 1961
SCALE AS SHOWN
SPEC. NO. DA-25-066-eng-
DRAWING NUMBER 18-04-01

THIS PLAN ACCOMPANIES CONTRACT No. DA-25-066-eng- MODIFICATION No.



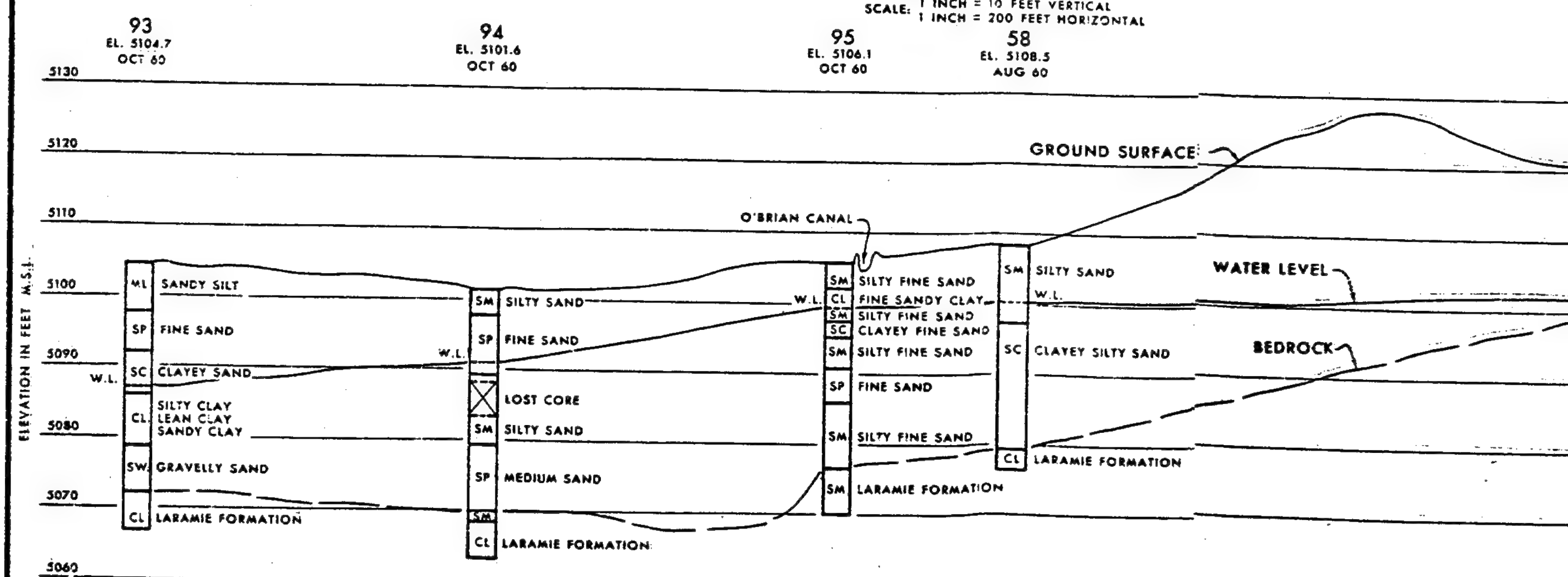
SECTION 2 - 2

SCALE: 1 INCH = 10 FEET VERTICAL
1 INCH = 200 FEET HORIZONTAL



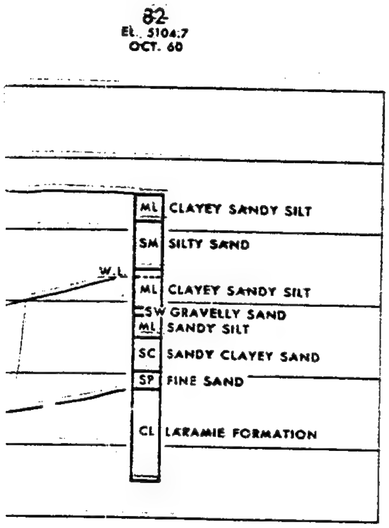
SECTION 3 - 3

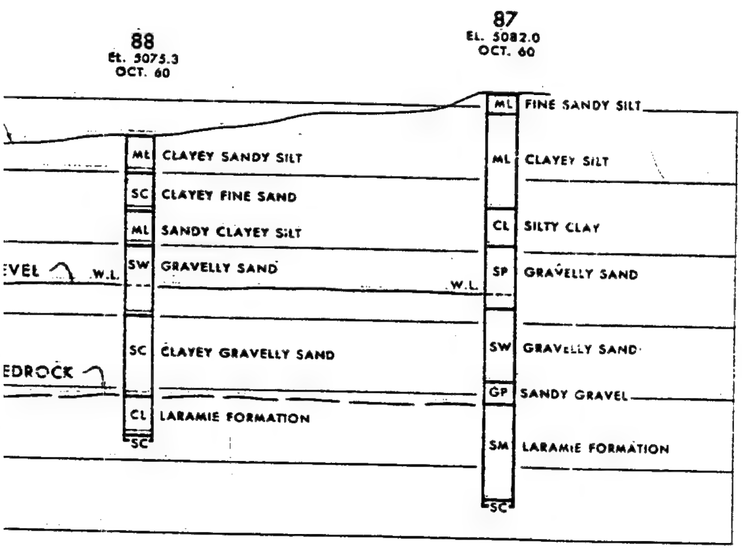
SCALE: 1 INCH = 10 FEET VERTICAL
1 INCH = 200 FEET HORIZONTAL

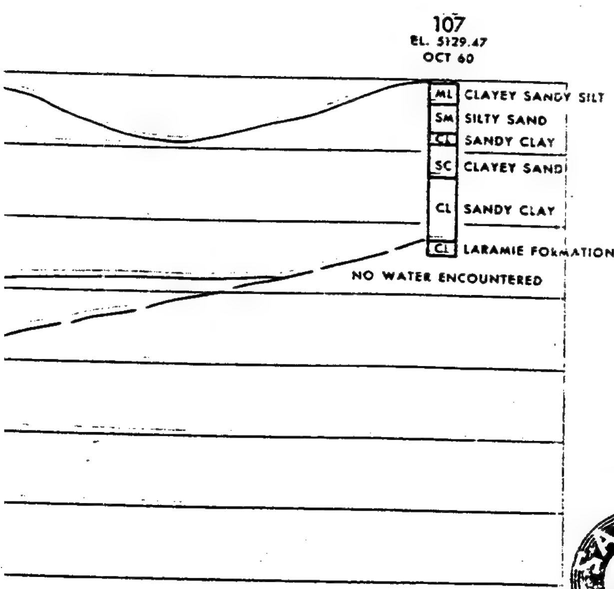


SECTION 4 - 4

SCALE: 1 INCH = 10 FEET VERTICAL
1 INCH = 200 FEET HORIZONTAL







REFERENCE DRAWINGS:

1. For locations of sections see dwg. no. 18-04-01, Plate 1

THIS DRAWING HAS BEEN REDUCED TO THREE EIGHTHS THE ORIGINAL SCALE

DATE	DESCRIPTION	MADE	APPRO

REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

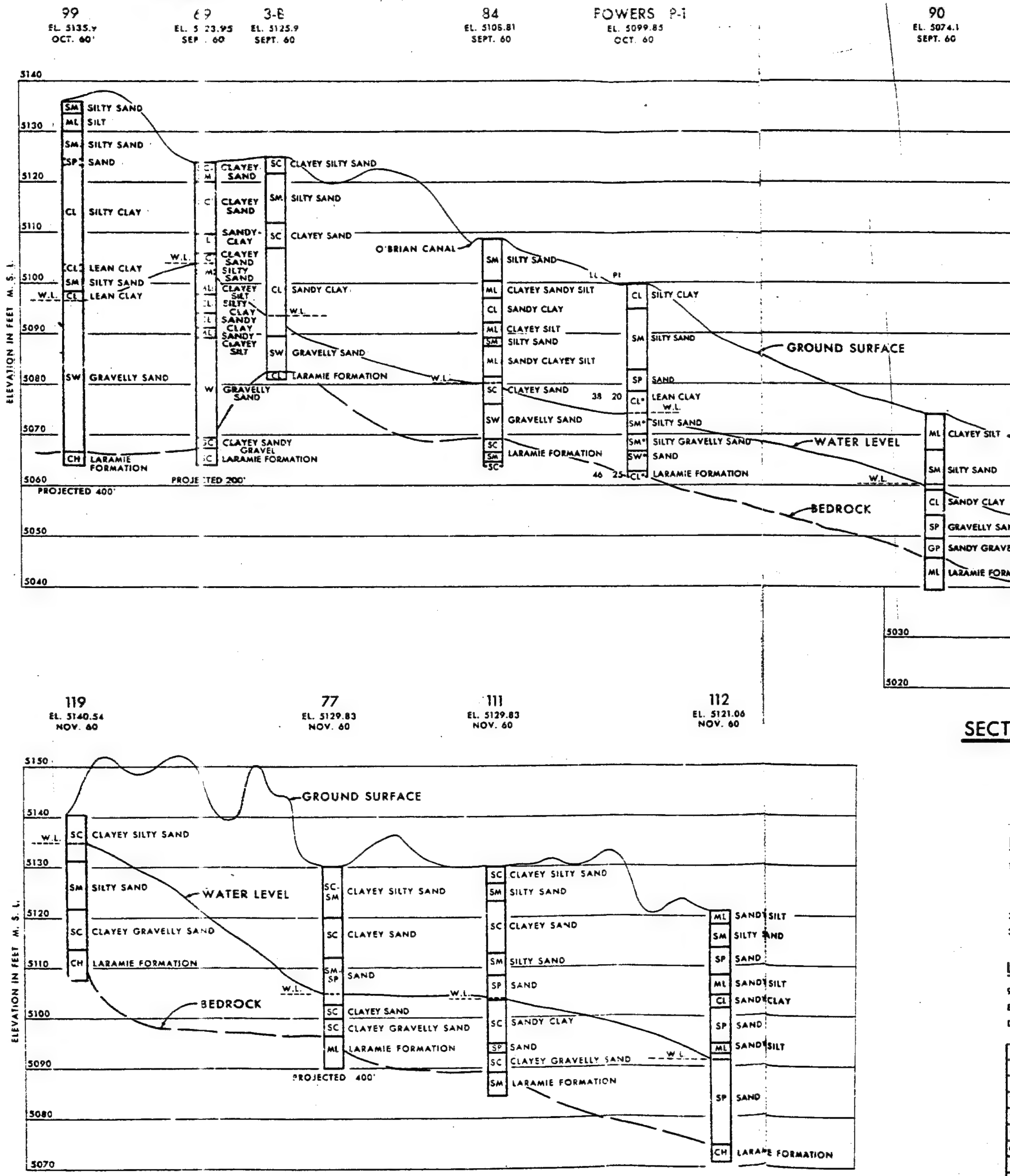
DESIGNED BY: C. W. H. / F. J. T.
DRAWN BY: C. R. S.
TRACED BY: C. R. S.
CHECKED BY: C. W. H. / F. J. T.
SUBMITTED BY: [signature]
APPROVED: H. A. Likes, CHIEF, FDN. & MAT'L. BRANCH
APPROVED: J. O. Ackerman, CHIEF, ENGINEERING DIVISION
APPROVED: H. G. Woodbury Jr., COL. C. E. DISTRICT ENGINEER

ROCKY MOUNTAIN ARSENAL — DENVER, COLORADO

PROGRAM FOR RECLAMATION
OF SURFACE AQUIFER
SECTIONS

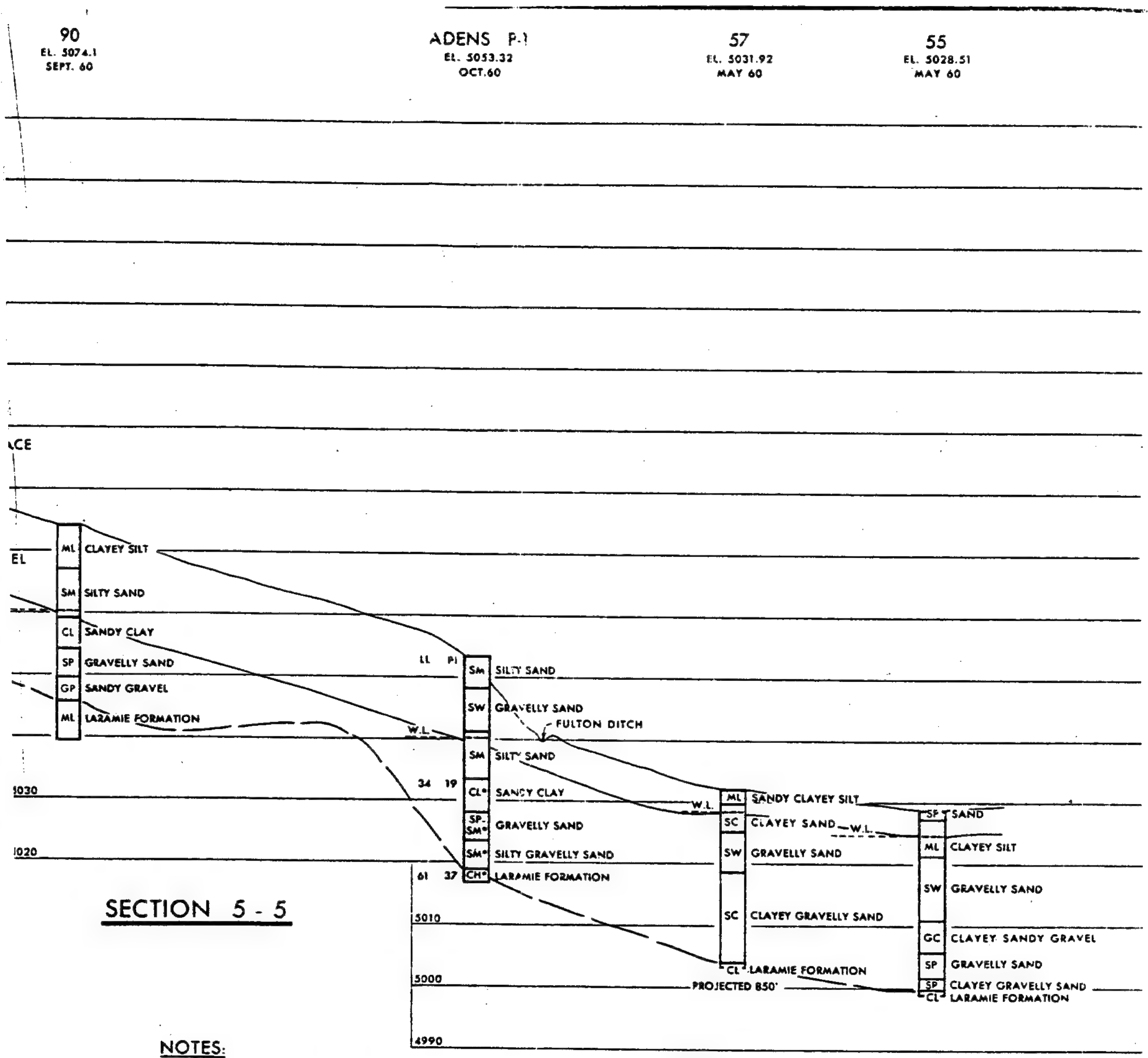
DATE: JAN. 1961
SCALE AS SHOWN
DRAWING NUMBER: 18-04-01

THIS PLAN ACCOMPANIES CONTRACT NO. DA-25-066-... MODIFICATION NO.



SECTION 6 - 6

SCALE: 1 INCH = 10 FEET VERTICAL
1 INCH = 600 FEET HORIZONTAL



SECTION 5 - 5

NOTES:

1. CLASSIFICATIONS AND DESCRIPTIONS AT THE RIGHT OF THE BORING PROFILE ARE SUPPLEMENTAL DATA BASED ON FIELD INSPECTION OF THE SAMPLES UNLESS OTHERWISE NOTED.
2. ALL ELEVATIONS REFER TO MEAN SEA LEVEL, 1929 GENERAL ADJUSTMENT.
3. SOIL CLASSIFIED BY THE UNIFIED SOIL CLASSIFICATION SYSTEM.

REFERENCE DRAWINGS:

1. For locations of sections see dwg. no. 18-04-01, Plate 1

LEGEND:

90 DRILL HOLE NUMBER
ELEV. 5108.81 ELEVATION TOP OF HOLE
DATE: MAY 60 DATE DRILLING COMPLETED

- * INDICATES LABORATORY CLASSIFICATION
- CL LEAN OR SANDY CLAY
- CH FAT OR SANDY FAT CLAY
- SP SAND, POORLY GRADED
- SW SAND, WELL GRADED
- GC GRAVEL, CLAYEY
- GM GRAVEL, SILTY
- SC SAND, CLAYEY
- SM SAND, SILTY
- GP GRAVEL, POORLY GRADED
- GW GRAVEL, WELL GRADED
- ML SILT, LOW PLASTICITY
- MH SILT, HIGH PLASTICITY
- LL LIQUID LIMIT
- PI PLASTICITY INDEX
- W.L. WATER LEVEL

THIS DRAWING HAS BEEN REDUCED TO THREE EIGHTHS THE ORIGINAL SCALE

DATE	DESCRIPTION	MADE	APPR'D

REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

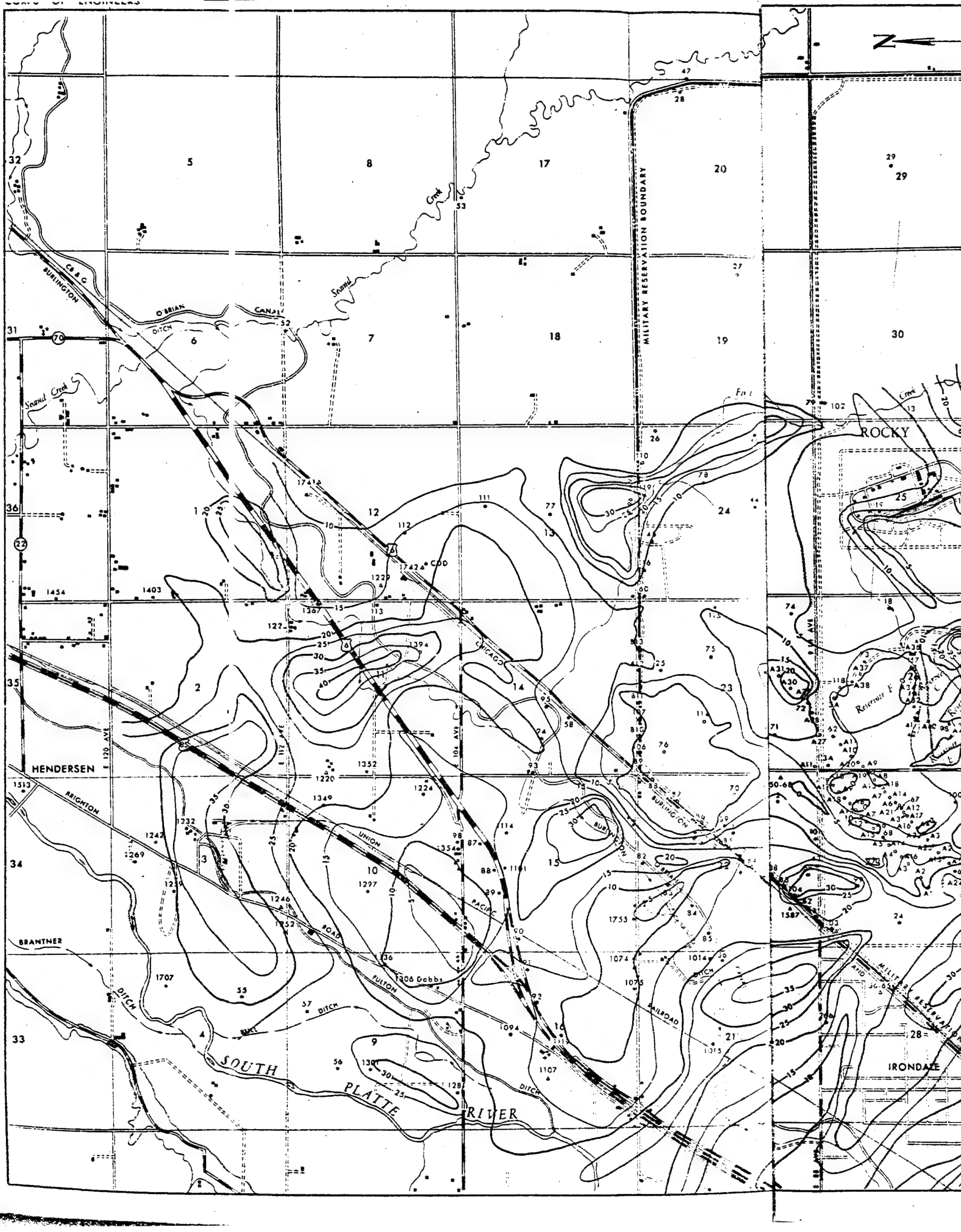
ROCKY MOUNTAIN ARSENAL — DENVER, COLORADO

PROGRAM FOR RECLAMATION OF SURFACE AQUIFER
SECTIONS

DESIGNED BY: C W H / F J T
DRAWN BY: C R S
TRACED BY: C R S
CHECKED BY: C W H / F J T
SUBMITTED BY: CHIEF, SPEC STUDY SECTION
APPROVED: CHIEF, FDN & MAT'L BRANCH
APPROVED: CHIEF, ENGINEERING DIVISION
DATE: JAN. 1961
APPROVED: COL, C. E. DISTRICT ENGINEER
SCALE AS SHOWN
SPEC. NO. DA-25-066-
DRAWING NUMBER 18-04-01

THIS PLAN ACCOMPANIED CONTRACT NO. DA-25-066- MODIFICATION NO.

PLATE 7



MILITARY RESERVATION BOUNDARY
Second Creek
O'BRIAN CANAL
DITCH
CB & Q
BURLINGTON
CHICAGO
ROCKY
Reservoir
COD
HENDERSEN
E 120 AVE
E 112 AVE
E 104 AVE
BRIGHTON
UNION
PACIFIC
ROAD
FULTON
DITCH
RAILROAD
BRANTNER
BULL
SOUTH
PLATTE
RIVER
Dobbs
IRONDALE

U. S. ARMY

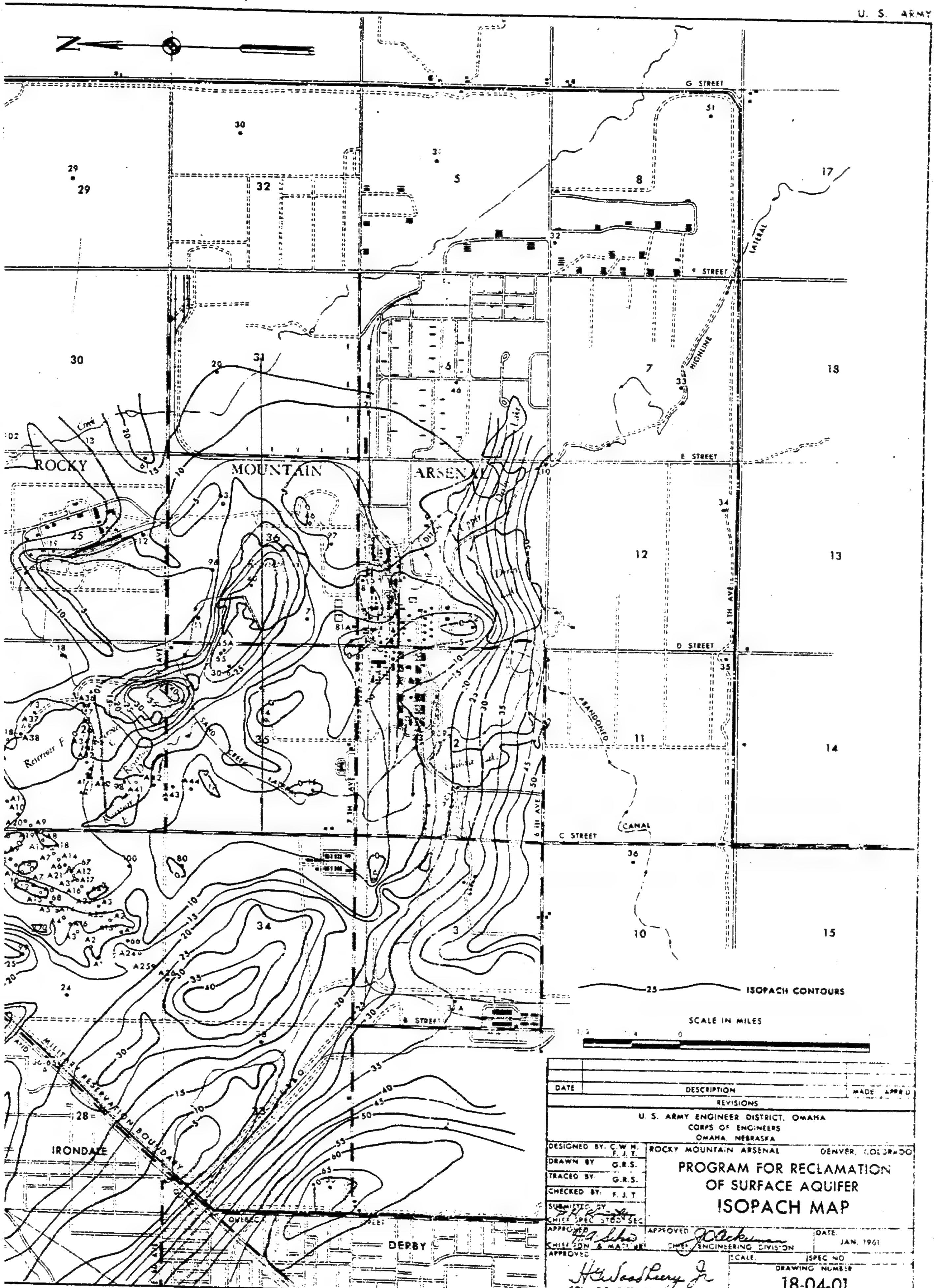
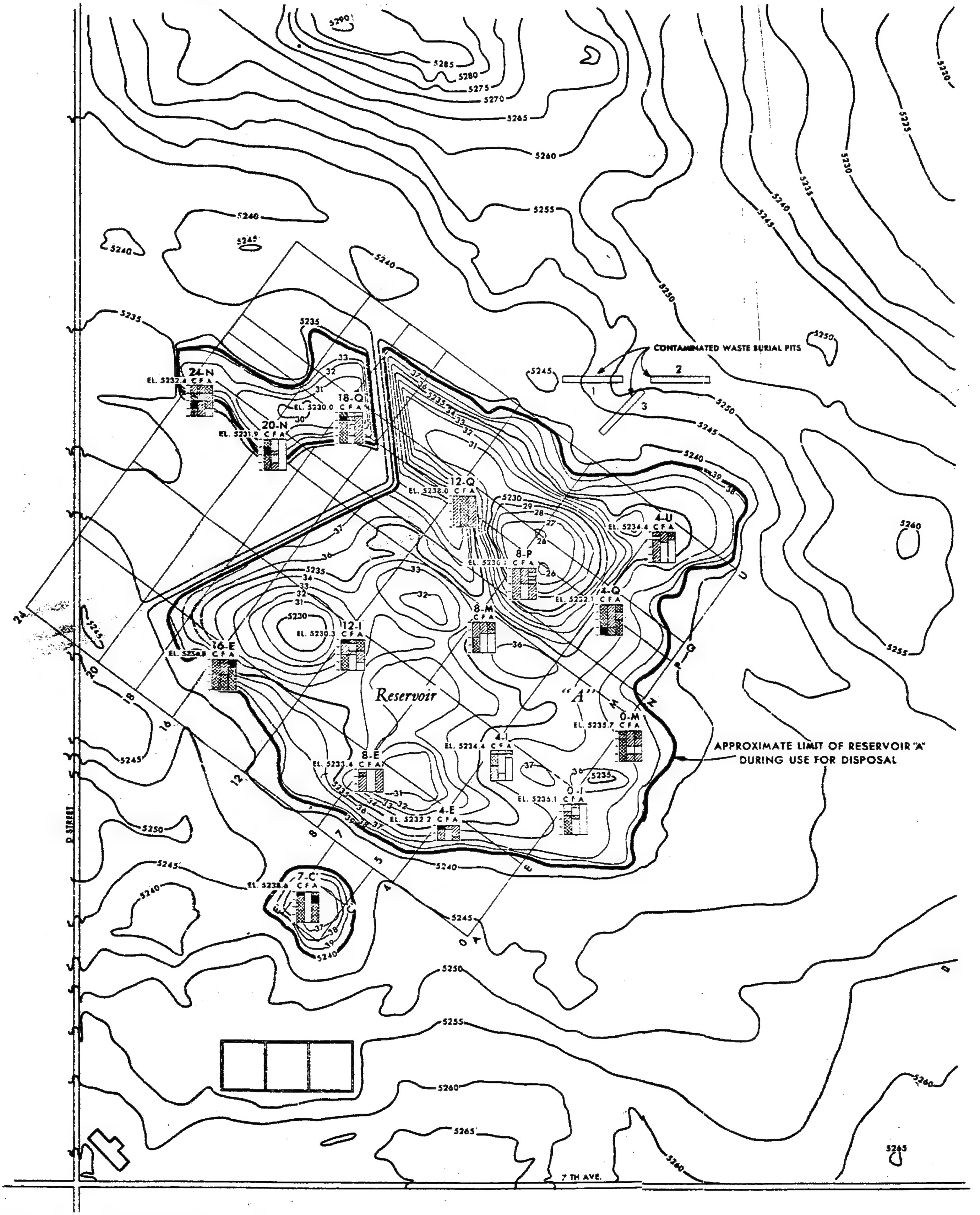
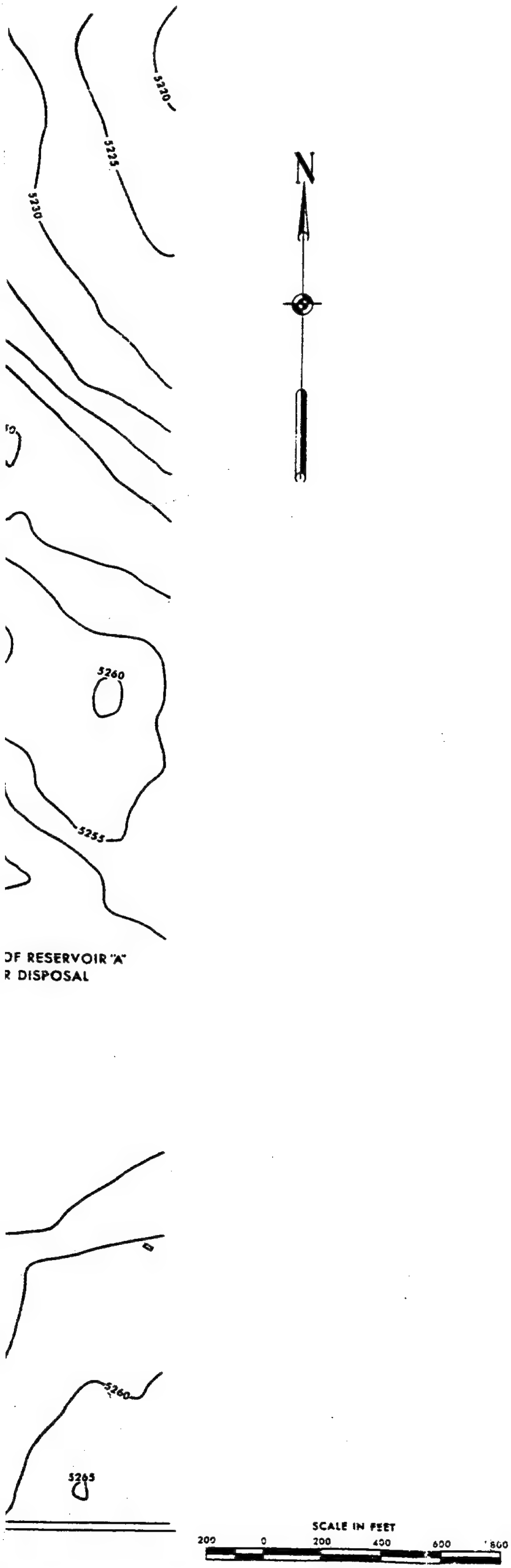


PLATE 9


CONTAMINATED WASTE BURIAL PITS
Reservoir
"A"
APPROXIMATE LIMIT OF RESERVOIR "A"
DURING USE FOR DISPOSAL
D STREET
7 TH AVE.
24-N
20-N
18-Q
12-Q
8-P
4-U
4-Q
16-E
12-I
8-M
8-E
4-I
0-M
0-I
4-E
7-C
EL. 5232.4
EL. 5231.9
EL. 5230.0
EL. 5238.0
EL. 5234.4
EL. 5230.3
EL. 5234.8
EL. 5233.4
EL. 5235.7
EL. 5235.1
EL. 5238.6



LEGEND:

CONTAMINATION (PPM)

DEPTH IN FEET	CHLORIDE	FLUORIDE	ARSENIC
1'	> 4000	> 150	> 10.0
2'	1001-4000	50.1-150	1.0-10.0
3'	100-1000	10.1-50	0.1-1.0
4'	< 100	< 10	0

(Legend columns: C — CHLORIDE, F — FLUORIDE, A — ARSENIC)

THIS DRAWING HAS BEEN REDUCED TO THREE EIGHTHS THE ORIGINAL SCALE

DATE	DESCRIPTION	MADE	APPR'D
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

DESIGNED BY: C.W.H. F.J.T.
DRAWN BY: G.R.S.
TRACED BY: G.R.S.
CHECKED BY: C. W. H. F. J. T.
SUBMITTED BY: CHIEF, SPEC. STDD. SEC.
APPROVED: H.G. Sikes CHIEF, FDN. & MATL. BR.
APPROVED: CHIEF ENGINEERING DIVISION
APPROVED: H.G. Woodbury Jr. COL., C.E. DISTRICT ENGINEER

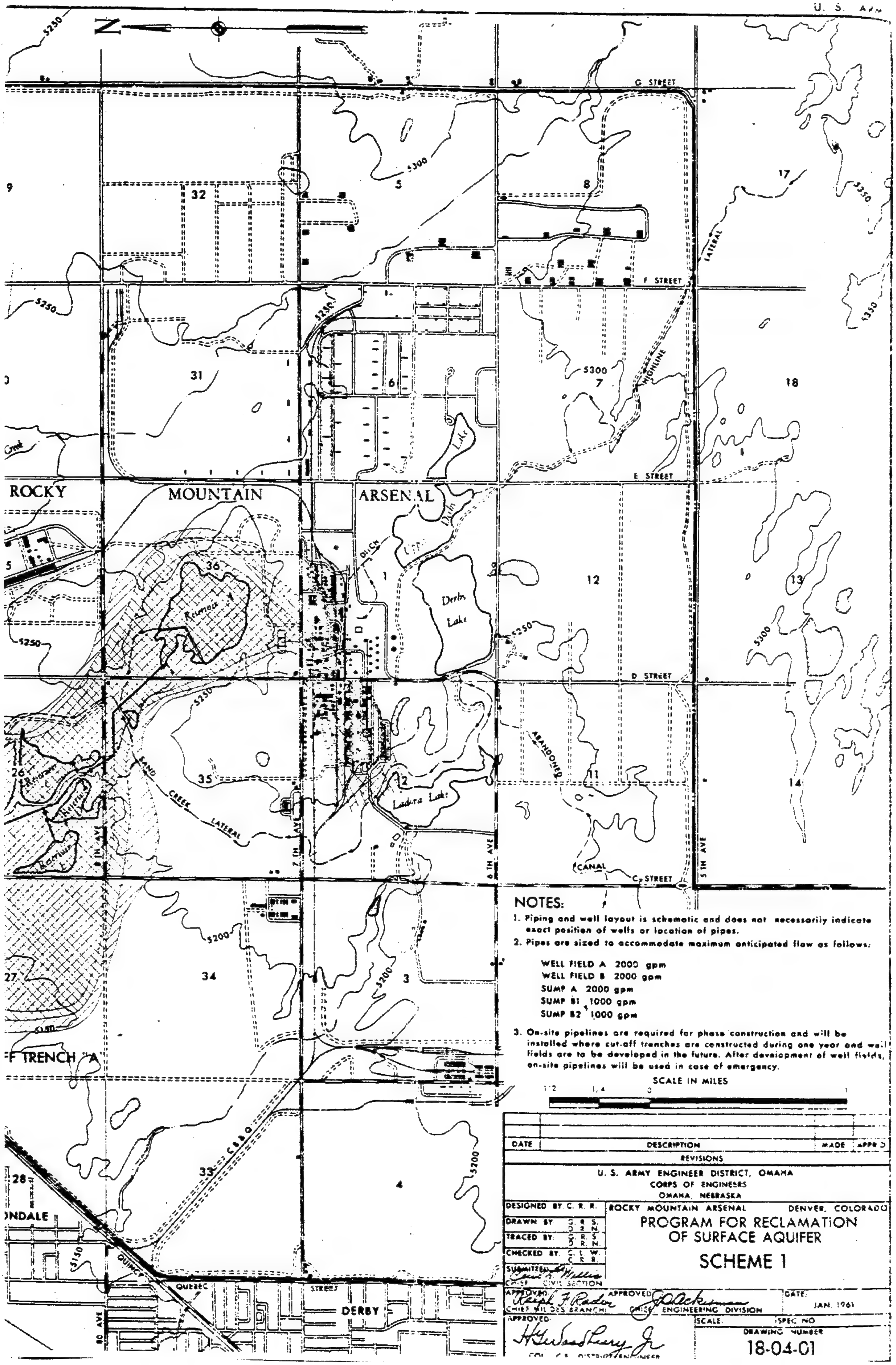
ROCKY MOUNTAIN ARSENAL — DENVER, COLORADO

PROGRAM FOR RECLAMATION OF SURFACE AQUIFER

RESERVOIR "A" CONTAMINATION

DATE: JAN 1961
SCALE:
SPEC. NO.
DRAWING NUMBER 18-04-01

WELL FIELD "B"
WELL FIELD "A"
ROCKY
SUMP B2
SUMP B1
SUMP A
CUT-OFF TRENCH
HENDERSEN
IRONDALE
SOUTH PLATTE RIVER
O'BRIAN CANAL
BURLINGTON
UNION PACIFIC
BRIGHTON
BRANTNER
BULL DITCH
FULTON DITCH
First Creek
Second Creek
LINE 20
LINE 21
LINE 22
LINE 23
LINE 24
LINE 25
LINE 26
LINE 16
LINE 14
LINE 13
LINE 9
LINE 5
LINE 1
LINE 2
E 120 AVE
E 112 AVE
E 104 AVE
CHICAGO
Plate 13



NOTES:

1. Piping and well layout is schematic and does not necessarily indicate exact position of wells or location of pipes.
2. Pipes are sized to accommodate maximum anticipated flow as follows:

WELL FIELD A 2000 gpm
WELL FIELD B 2000 gpm
SUMP A 2000 gpm
SUMP B1 1000 gpm
SUMP B2 1000 gpm

3. On-site pipelines are required for phase construction and will be installed where cut-off trenches are constructed during one year and well fields are to be developed in the future. After development of well fields, on-site pipelines will be used in case of emergency.

SCALE IN MILES

1/2 1/4 0 1

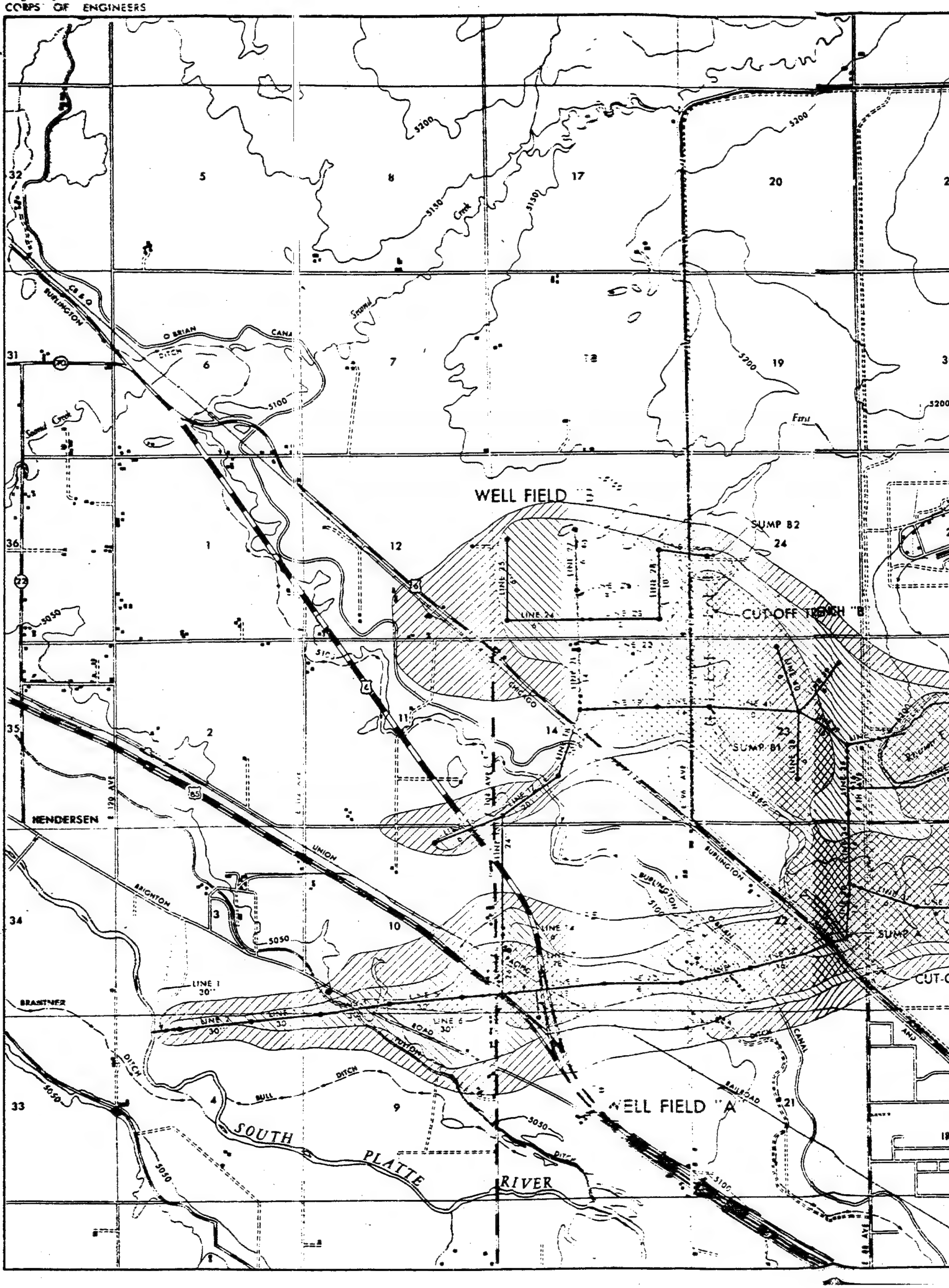
DATE	DESCRIPTION	MADE	APPR D

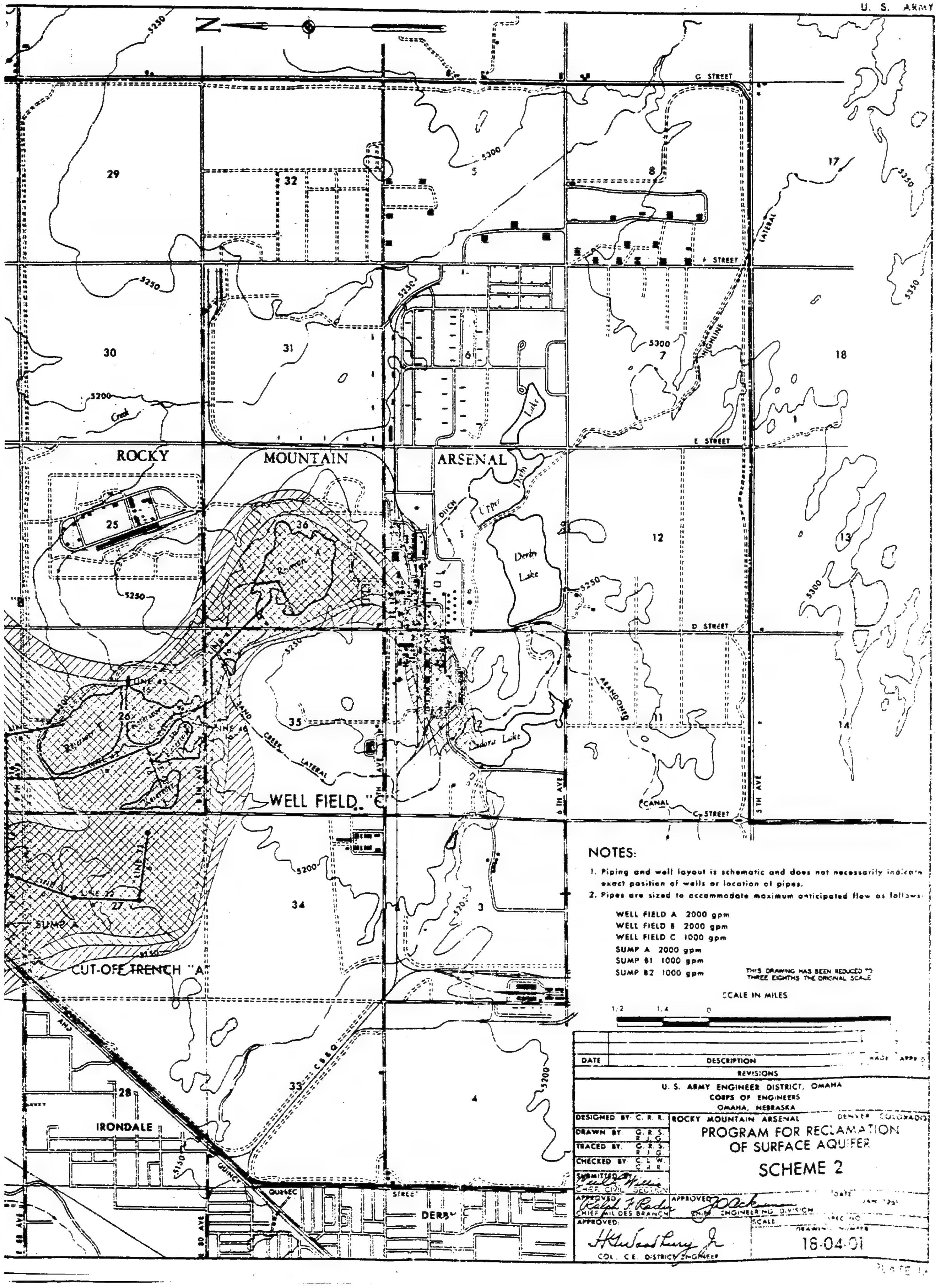
REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

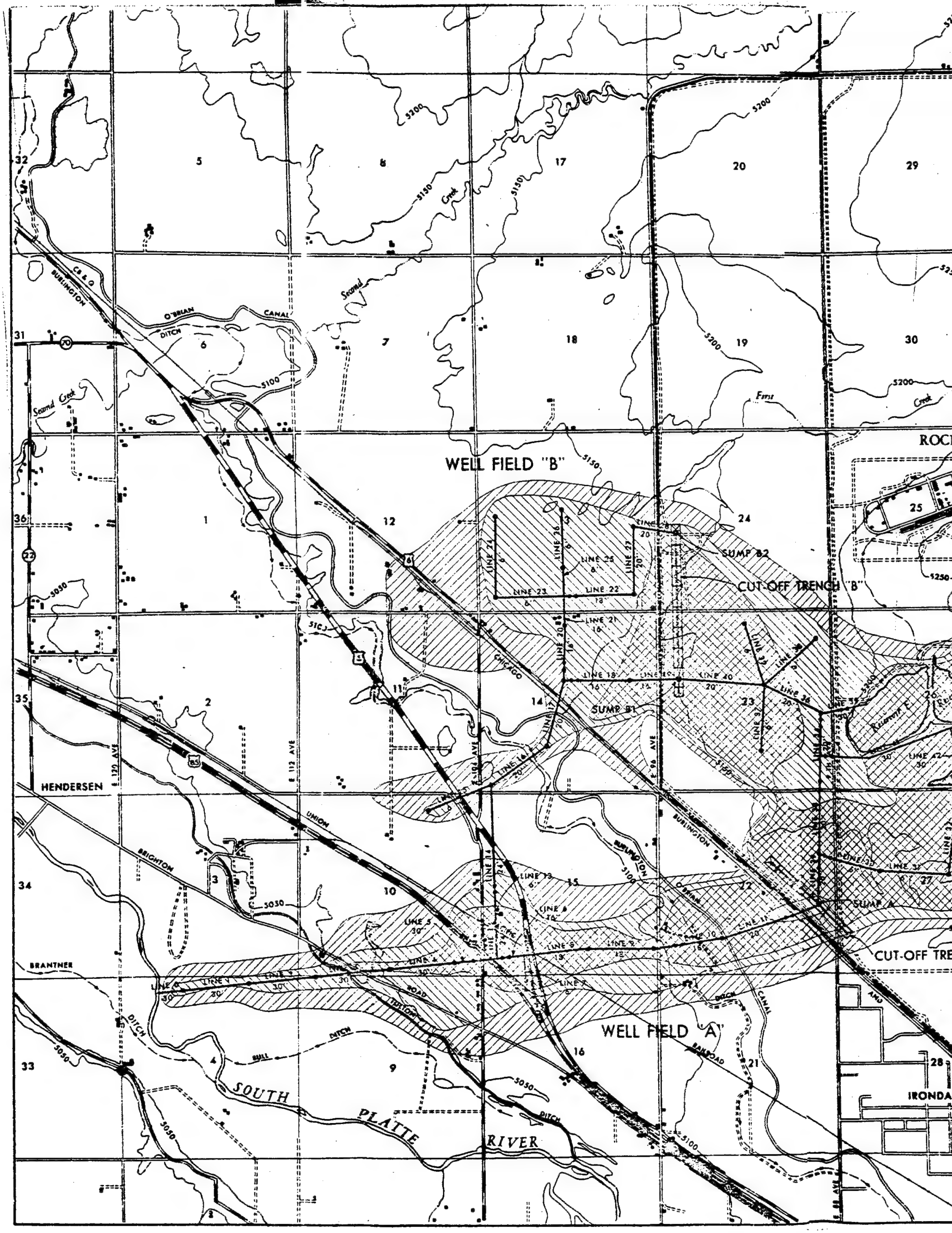
DESIGNED BY C. R. R.	ROCKY MOUNTAIN ARSENAL DENVER, COLORADO
DRAWN BY G. R. S. / J. R. N.	PROGRAM FOR RECLAMATION OF SURFACE AQUIFER
TRACED BY G. R. S. / J. R. N.	
CHECKED BY C. L. W. / C. R. R.	SCHEME 1
SUBMITTED [signature] CHIEF CIVIL SECTION	
APPROVED: [signature] CHIEF MIL DES BRANCH	APPROVED: [signature] CHIEF ENGINEERING DIVISION	DATE: JAN. 1961
APPROVED: [signature] COL. C.E. DISTRICT ENGINEER	SCALE: SPEC NO DRAWING NUMBER 18-04-01

CORPS OF ENGINEERS





U. S. ARMY
N
G STREET
F STREET
E STREET
D STREET
C STREET
ROCKY MOUNTAIN ARSENAL
29
32
5
8
17
30
31
6
7
18
25
36
12
13
26
35
2
11
14
27
34
3
33
4
28
5300
5250
5200
5150
5350
Creek
HIGHLINE
LATERAL
Upper
Ditch
Derby Lake
Ladora Lake
ABANDONED
CANAL
SAND CREEK LATERAL
WELL FIELD "C"
SUMP A
CUT-OFF TRENCH "A"
"B"
LINE 45
LINE 46
LINE 32
LINE 33
Reservoir F
Reservoir C
Reservoir A
C B & Q
IRONDALE
QUEBEC
QUINCY
DERBY
STREET
5 TH AVE
6 TH AVE
7 TH AVE
8 TH AVE
80 AVE
88 AVE
NOTES:
1. Piping and well layout is schematic and does not necessarily indicate exact position of wells or location of pipes.
2. Pipes are sized to accommodate maximum anticipated flow as follows:
WELL FIELD A 2000 gpm
WELL FIELD B 2000 gpm
WELL FIELD C 1000 gpm
SUMP A 2000 gpm
SUMP B1 1000 gpm
SUMP B2 1000 gpm
THIS DRAWING HAS BEEN REDUCED TO THREE EIGHTHS THE ORIGINAL SCALE
SCALE IN MILES
1/2
1/4
0
DATE
DESCRIPTION
REVISIONS
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA
DESIGNED BY: C. R. R.
DRAWN BY: G. R. S. R. J. G.
TRACED BY: G. R. S. R. J. G.
CHECKED BY: C. L. W. C. R. R.
SUBMITTED BY:
CHIEF, CIVIL SECTION
APPROVED:
Ralph F. Rader
CHIEF, MIL DES BRANCH
APPROVED
CHIEF, ENGINEERING DIVISION
APPROVED:
COL. C.E. DISTRICT ENGINEER
ROCKY MOUNTAIN ARSENAL
DENVER COLORADO
PROGRAM FOR RECLAMATION OF SURFACE AQUIFER
SCHEME 2
DATE
JAN 1951
SCALE
SPEC NO
DRAWING NUMBER
18-04-01
PLATE 1A



WELL FIELD "B"
WELL FIELD "A"
CUT-OFF TRENCH "B"
SUMP B2
SUMP B1
SUMP A
HENDERSEN
SOUTH PLATTE RIVER
BRANTNER
BRIGHTON
UNION
BURLINGTON
CHICAGO
O'BRIAN DITCH
CANAL
Second Creek
First Creek
FULTON
BULL
DITCH
RAILROAD
E 120 AVE
E 112 AVE
E 104 AVE
E 96 AVE
E 88 AVE
LINE 23
LINE 22
LINE 24
LINE 25
LINE 26
LINE 27
LINE 8
LINE 21
LINE 20
LINE 18
LINE 40
LINE 14
LINE 13
LINE 5
LINE 2
LINE 9
LINE 7
LINE 6
LINE 42
5200
5150
5100
5050
5250

U. S. ARMY

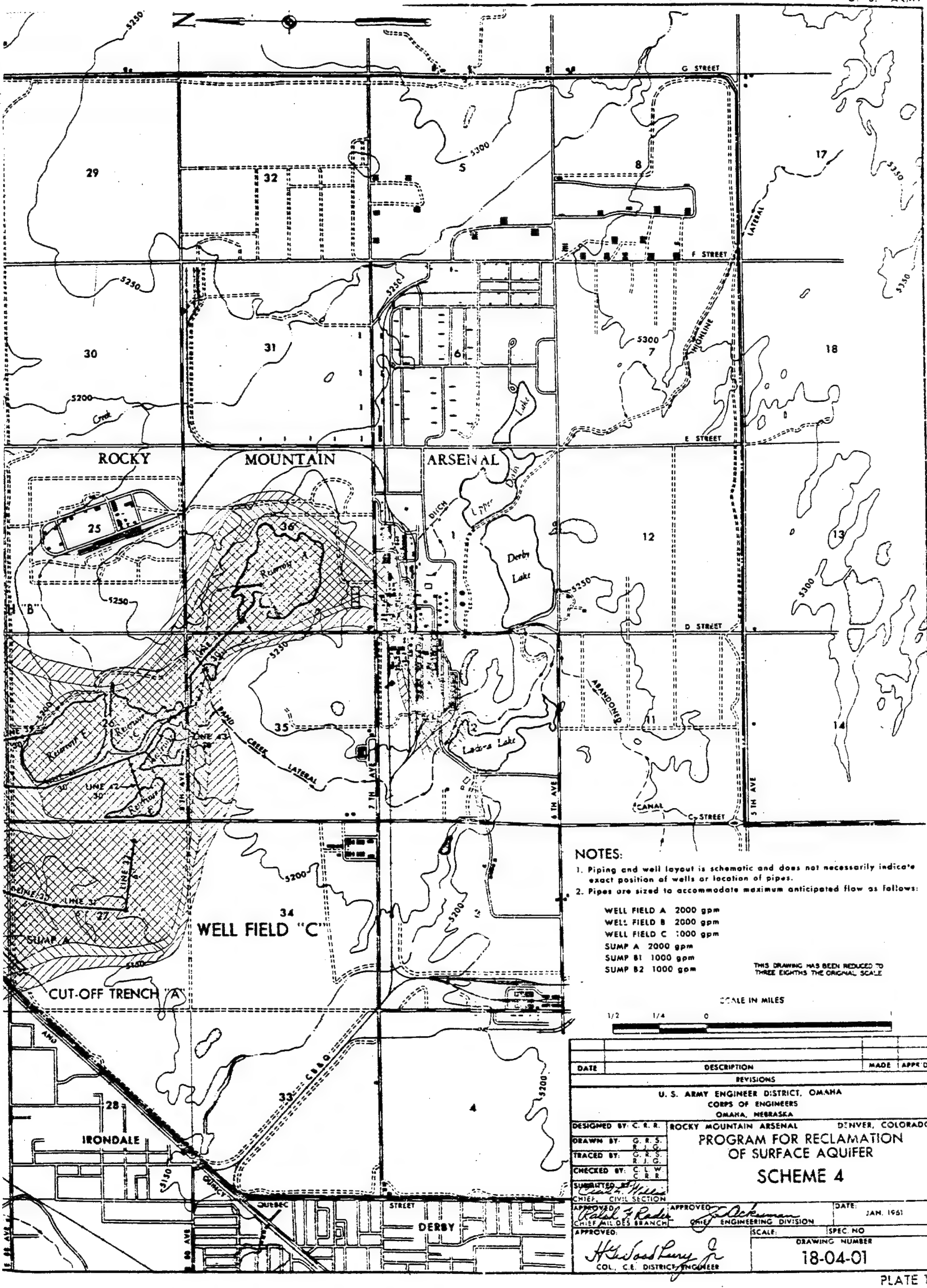
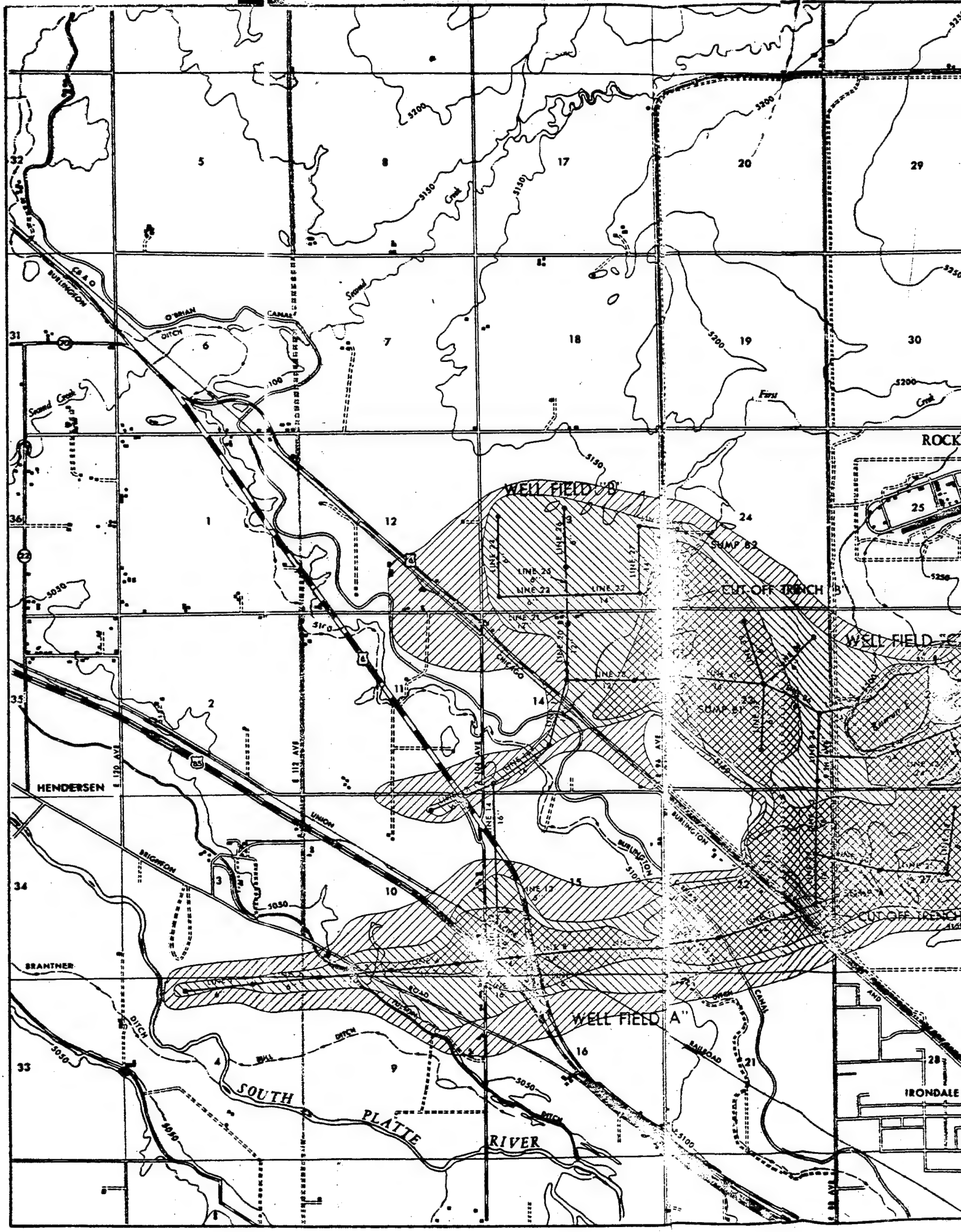
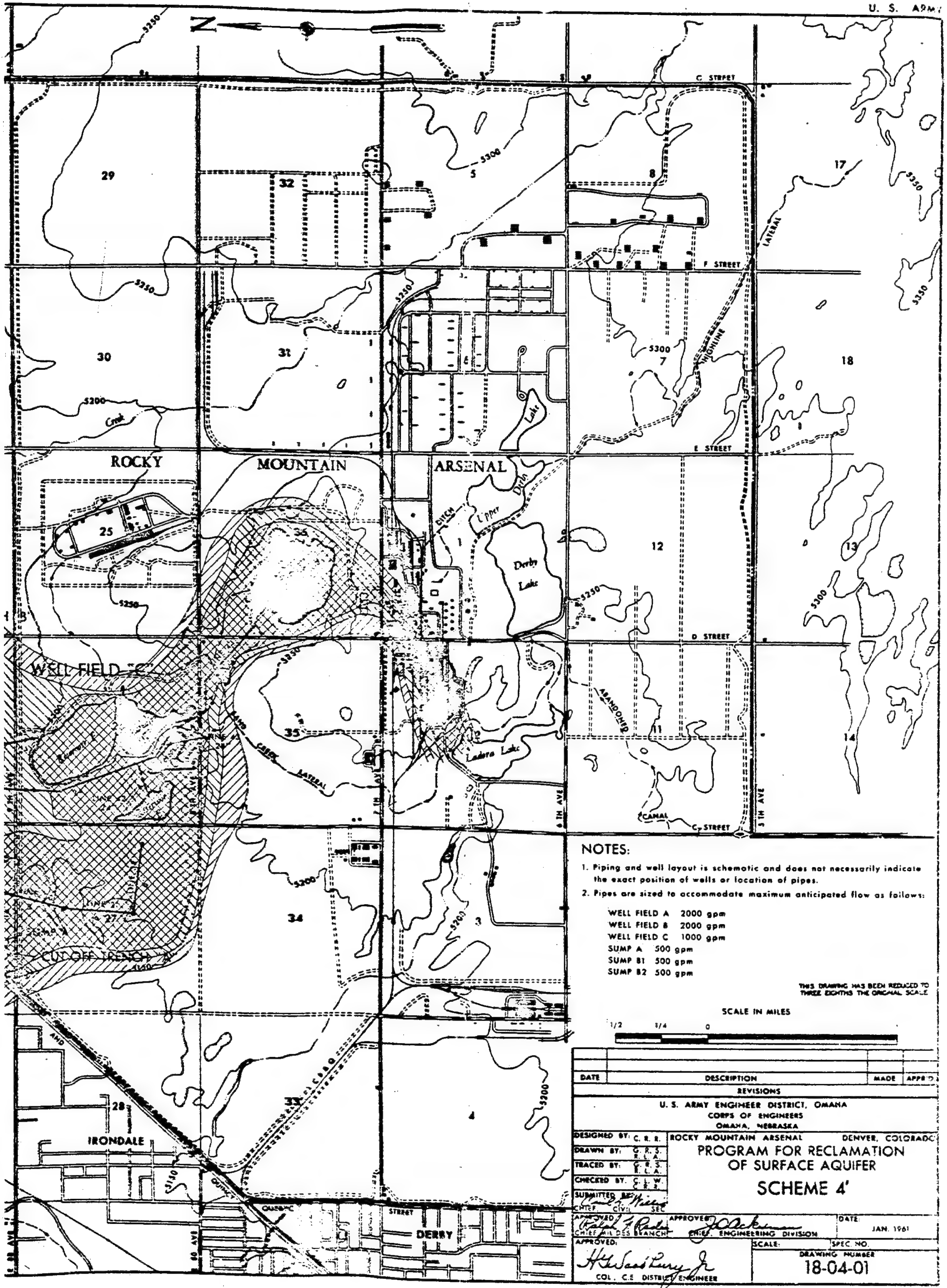


PLATE 15


WELL FIELD "B"
SUMP B2
CUT-OFF TRENCH "B"
WELL FIELD "C"
SUMP B1
WELL FIELD "A"
SOUTH PLATTE RIVER
HENDERSEN
IRONDALE
ROCK
BURLINGTON
UNION
BRIGHTON
CHICAGO
BRANTNER
O'BRIAN CANAL
Second Creek
First Creek
E 120 AVE
E 112 AVE
E 104 AVE
E 96 AVE
E 88 AVE
5200
5150
5100
5050
5250

U. S. ARMY



NOTES:

1. Piping and well layout is schematic and does not necessarily indicate the exact position of wells or location of pipes.
2. Pipes are sized to accommodate maximum anticipated flow as follows:

WELL FIELD A 2000 gpm
WELL FIELD B 2000 gpm
WELL FIELD C 1000 gpm
SUMP A 500 gpm
SUMP B1 500 gpm
SUMP B2 500 gpm

THIS DRAWING HAS BEEN REDUCED TO THREE EIGHTHS THE ORIGINAL SCALE

SCALE IN MILES

1/2 1/4 0

DATE	DESCRIPTION	MADE	APPR'D.

REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

DESIGNED BY: C. R. R.
DRAWN BY: G. R. S. R. L. A.
TRACED BY: G. R. S. R. L. A.
CHECKED BY: C. L. W. C. R. Z.
SUBMITTED BY: Carl ? Willis
CHIEF, CIVIL SEC

ROCKY MOUNTAIN ARSENAL DENVER, COLORADO

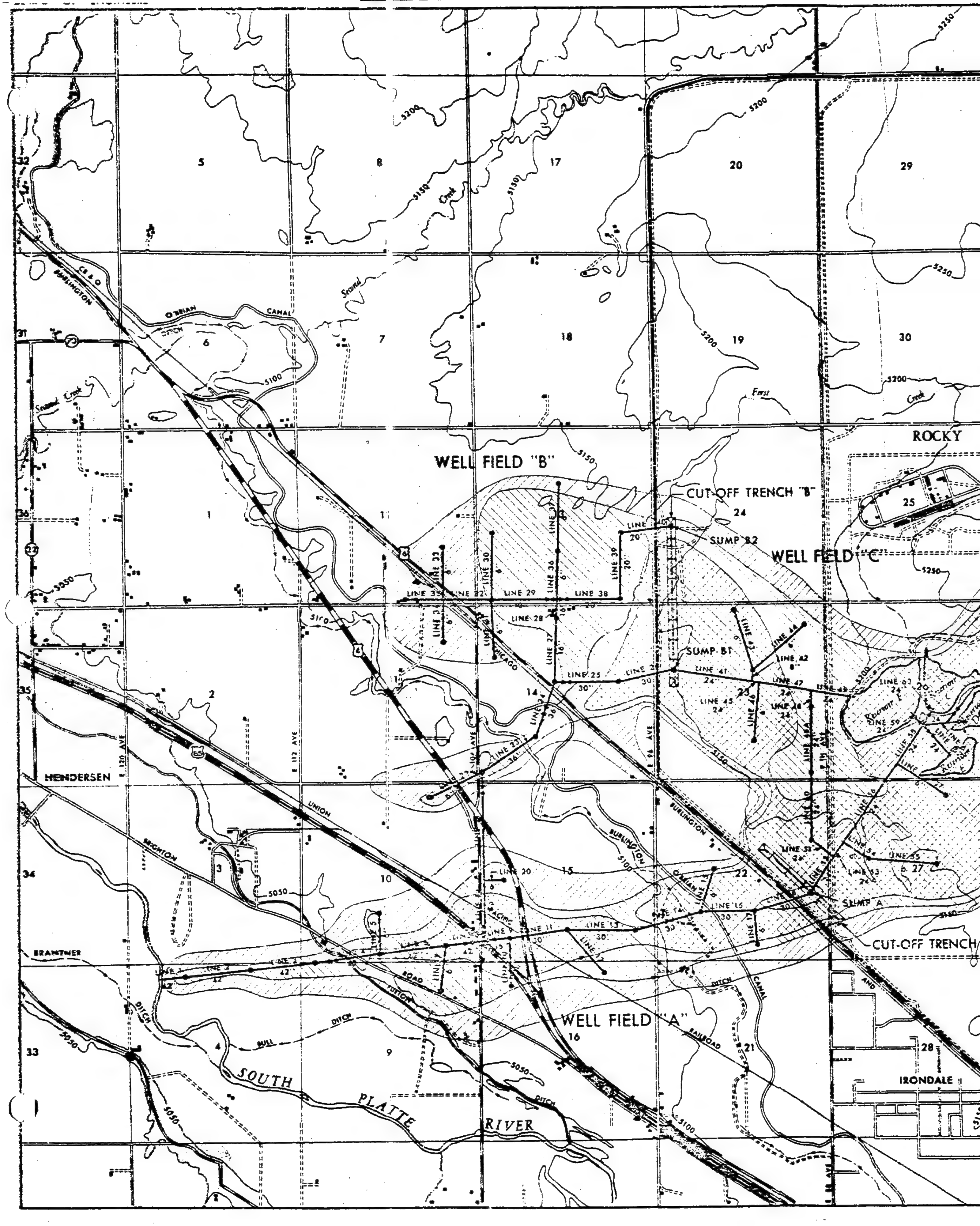
PROGRAM FOR RECLAMATION OF SURFACE AQUIFER

SCHEME 4'

APPROVED: Ralph F. Rader
CHIEF MIL DES BRANCH
APPROVED: J. Ackerman
CHIEF, ENGINEERING DIVISION
DATE: JAN. 1961

APPROVED: H. G. Woodbury Jr.
COL., C.E. DISTRICT ENGINEER
SCALE:
SPEC. NO.
DRAWING NUMBER 18-04-01

PLATE 16



WELL FIELD "A"
WELL FIELD "B"
WELL FIELD "C"
CUT-OFF TRENCH "B"
CUT-OFF TRENCH
SUMP B2
SUMP B1
SUMP A
ROCKY
HENDERSEN
IRONDALE
SOUTH PLATTE RIVER
BURLINGTON
CB & Q
UNION
PACIFIC
RAILROAD
CHICAGO
BRIGHTON
ROAD
FULTON
DITCH
BULL DITCH
BRANTNER
O'BRIAN CANAL
O'BRIAN DITCH
Second Creek
First Creek
E 120 AVE
E 112 AVE
E 104 AVE
E 96 AVE
E 88 AVE
5050
5100
5150
5200
5250
LINE 1
LINE 2
LINE 11
LINE 15
LINE 20
LINE 25
LINE 27
LINE 28
LINE 29
LINE 30
LINE 33
LINE 35
LINE 36
LINE 38
LINE 39
LINE 40
LINE 41
LINE 42
LINE 43
LINE 44
LINE 45
LINE 46
LINE 47
LINE 53
LINE 54
LINE 55
LINE 59
5
8
17
20
29
6
7
18
19
30
1
24
25
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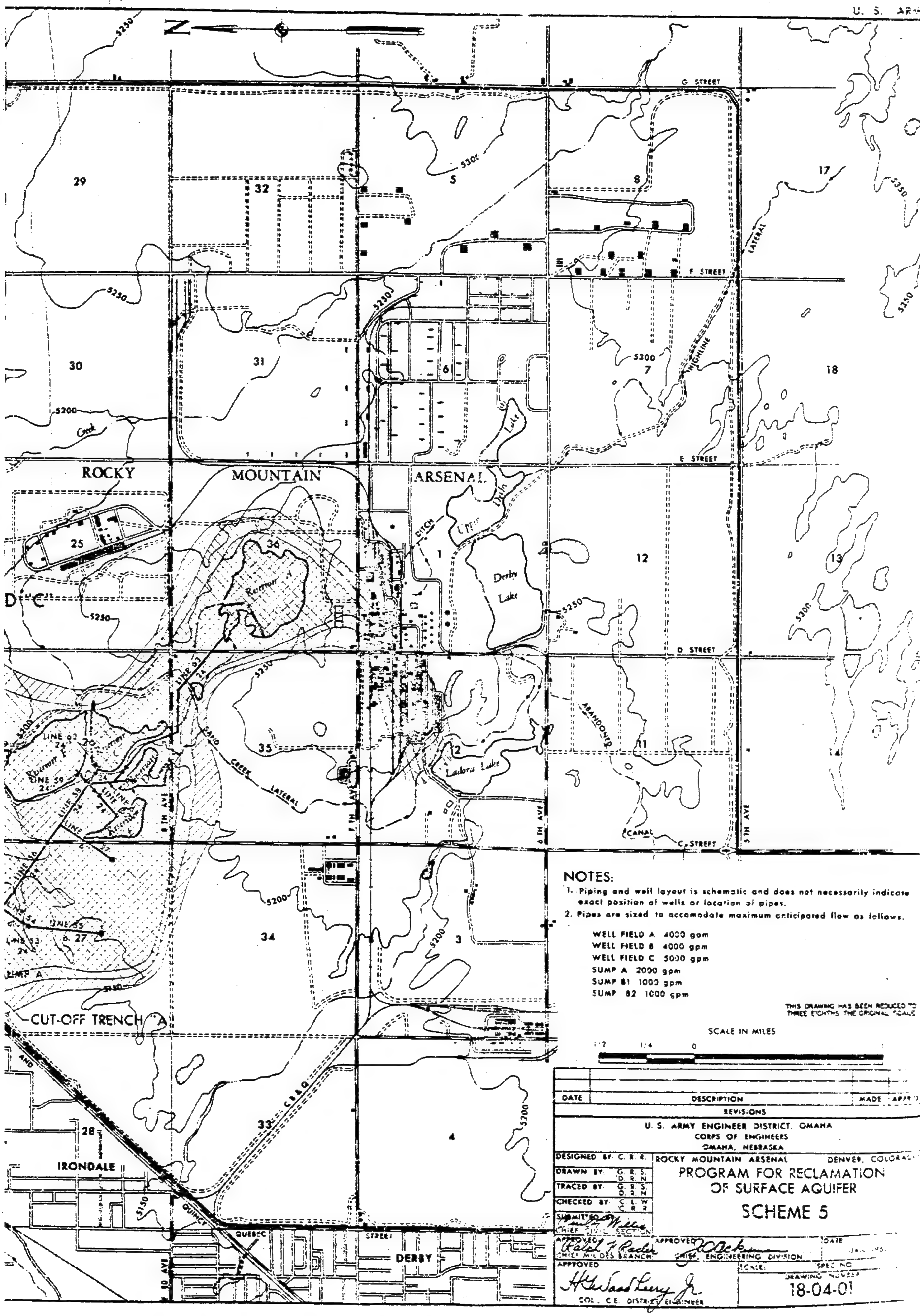
28
31
32
33
34
35
36



NOTES:

1. Piping and well layout is schematic and does not necessarily indicate exact position of wells or location of pipes.
2. Pipes are sized to accomodate maximum anticipated flow as follows:

WELL FIELD A 4000 gpm
WELL FIELD B 4000 gpm
WELL FIELD C 5000 gpm
SUMP A 2000 gpm
SUMP B1 1000 gpm
SUMP B2 1000 gpm

THIS DRAWING HAS BEEN REDUCED TO THREE EIGHTHS THE ORIGINAL SCALE

SCALE IN MILES

1/2 1/4 0 1

DATE	DESCRIPTION	MADE	APPR'D
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

DESIGNED BY: C. R. R.
DRAWN BY: G. R. S. / D. R. N.
TRACED BY: G. R. S. / D. R. N.
CHECKED BY: C. L. W. / C. R. R.

ROCKY MOUNTAIN ARSENAL — DENVER, COLORADO

PROGRAM FOR RECLAMATION OF SURFACE AQUIFER

SCHEME 5

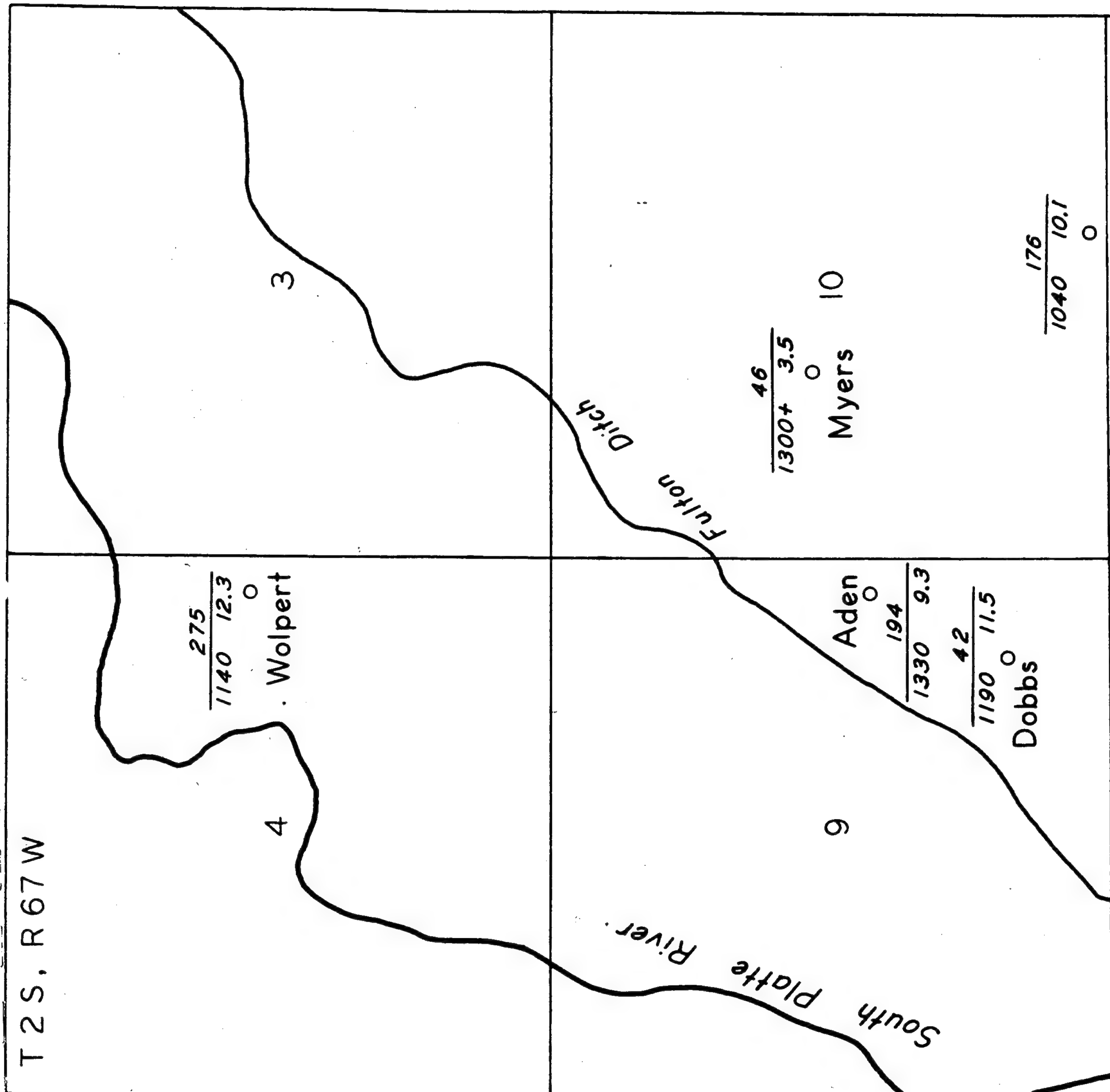
SUBMITTED: Paul F. Willis — CHIEF, CIVIL SECTION
APPROVED: Ralph F. Rader — CHIEF, MIL DES BRANCH
APPROVED: [signature] — CHIEF, ENGINEERING DIVISION
APPROVED: H. Gerwood Lacey Jr. — COL., C.E. DISTRICT ENGINEER

DATE: JAN. 195
SCALE:
SPEC NO
DRAWING NUMBER
18-04-01

MAP OF PART OF CONTAMINATED

ARSENAL SUMMARI

Drawdown-Rec

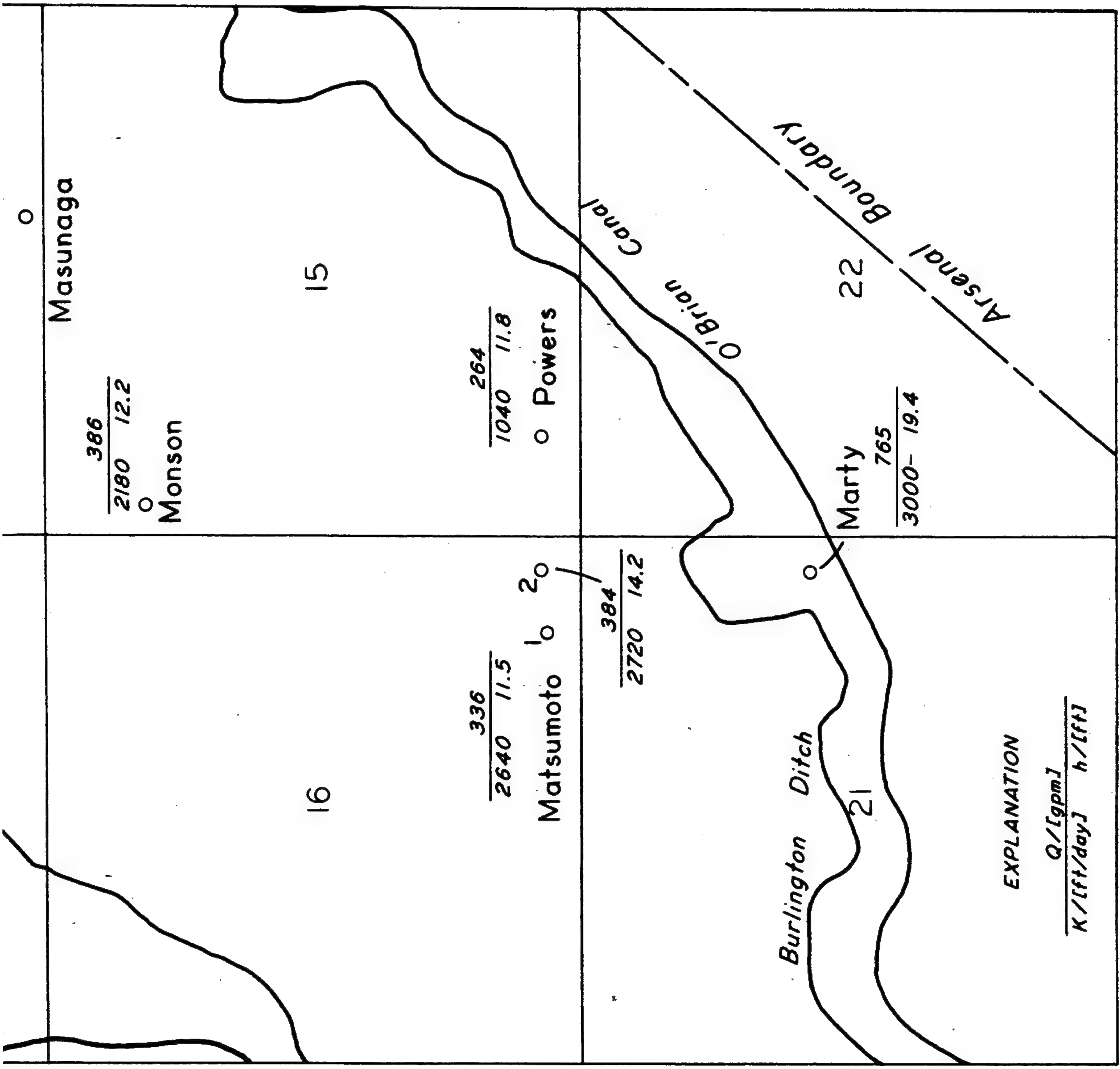


'ED AREA NEAR ROCKY MOUNTAIN

ARIZING WELL TESTS

Recovery Tests

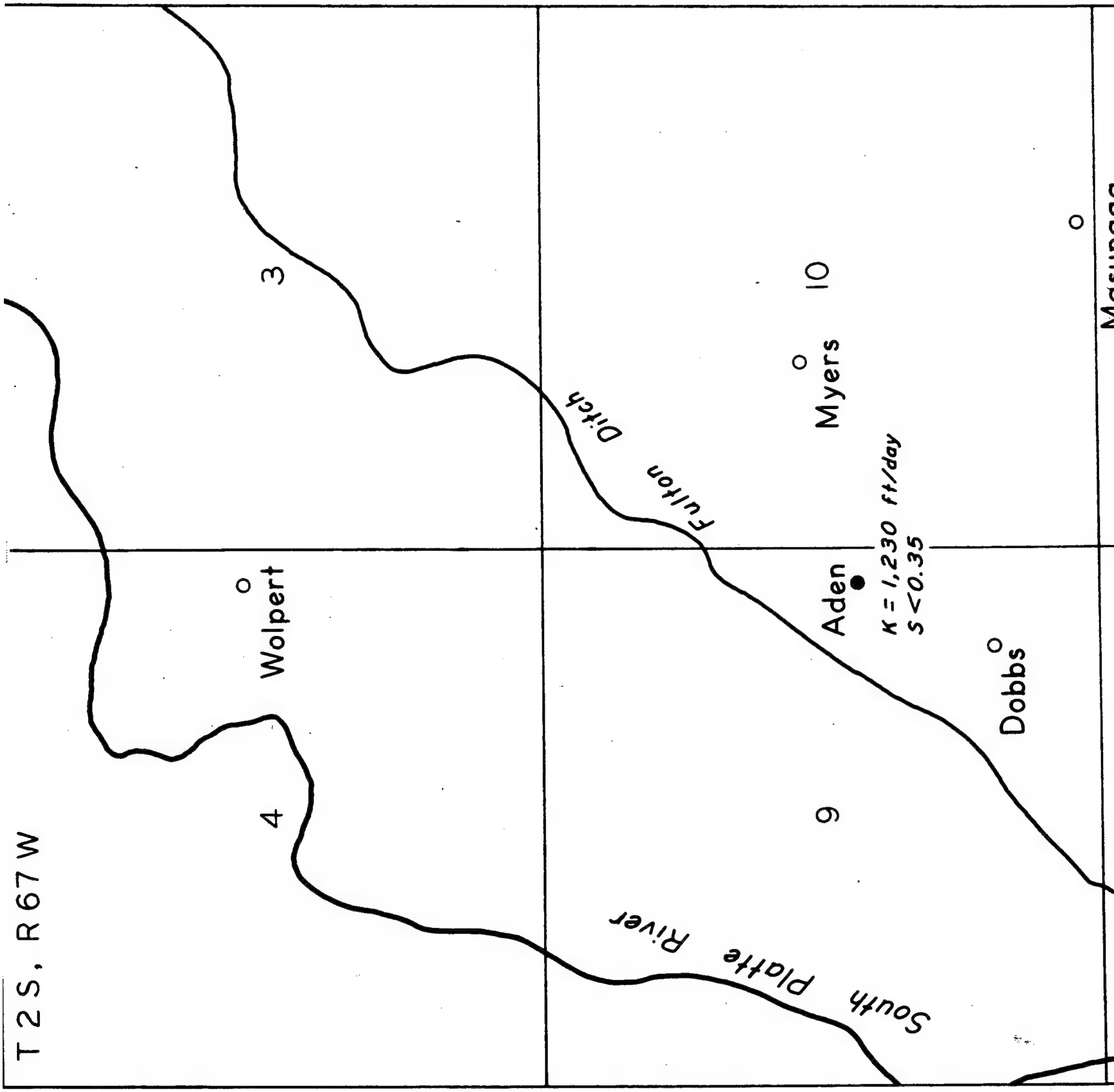
FIG. D-1a



MAP OF PART OF CONTAMINATED A

ARSENAL SUMMARIZ

Interfere



D AREA NEAR ROCKY MOUNTAIN
.RIZING WELL TESTS
erence Tests

FIG. D-1b

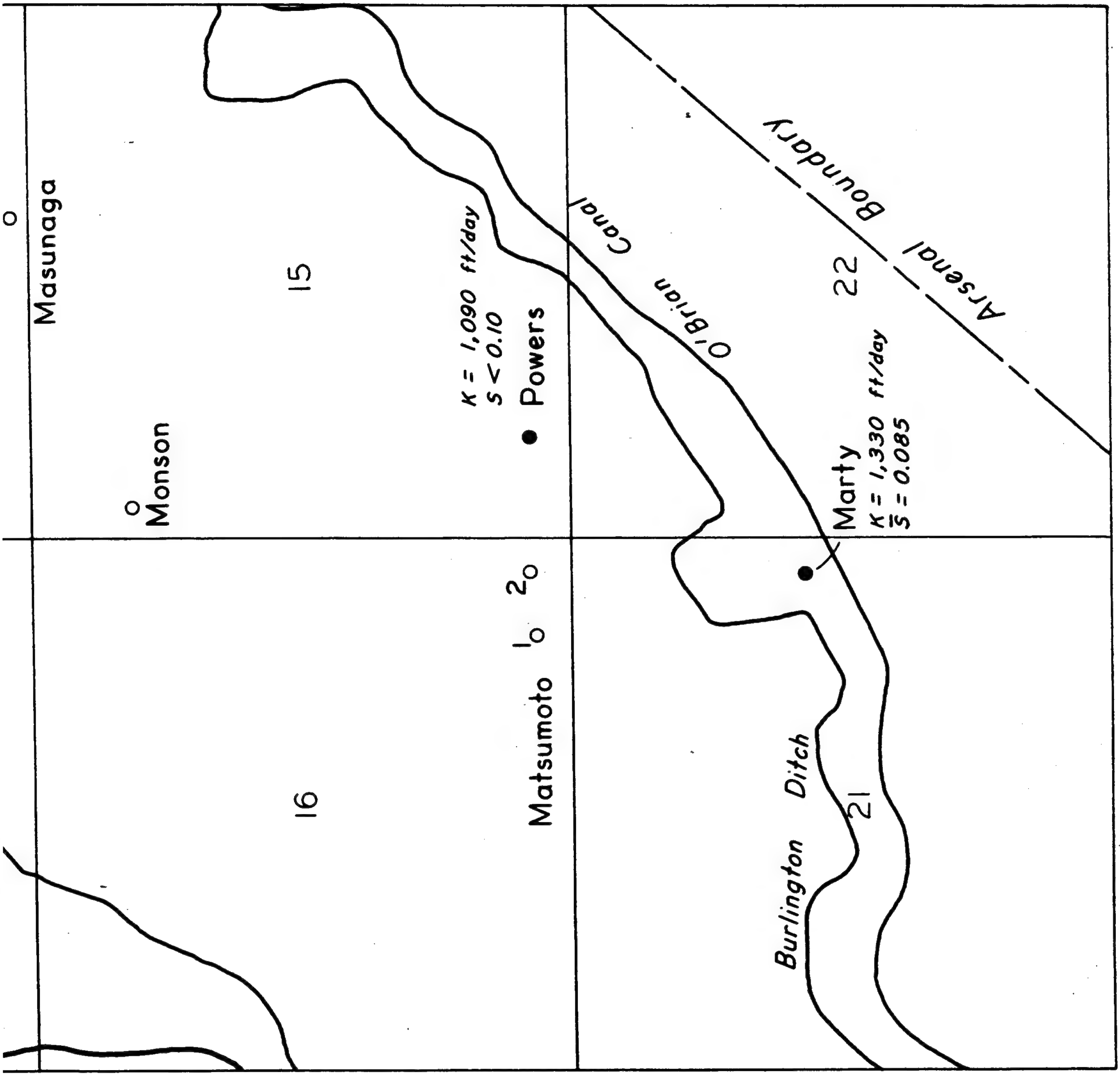


TABLE D-1

SUMMARY OF DRAWDOWN-RECOVERY TESTS OF WELLS

[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]	[10]
					Pump			Measured discharge	
Well name	Location 2S 67W	Depth,[a] feet	Casing diam. inches	Kind	Diam. inches	Horse power	Reported discharge, gpm	gpm	10^3 ft³/day
Aden	9 daa	24.1	48	concr.	-	10	400	194	37.4
Dobbs	9 ddb	45	48/24	steel	5	--	---	42	8.1
Marty	21 add	52	48	concr.	8	20	---	765*	147.0*
Masunaga	10 dcc	39	48	concr.	6	5	300	176	33.9
Matsumoto	16 ddc	36.9	48	concr.	6	5	400	336	64.7
Matsumoto	16 ddd2	36.94	48	concr.	6	5	500	384*	74.0*
Monson	15 badd	40	18	steel	6	10	400	386	74.4
Myers	10 bdcd	39.5	48	--	3	3	150	46	8.9
Powers	15 ccd	40	48	concr.	6	10	800	264	50.9
Wolpert	4 adb	18.1	36	concr.	--	--	250	275	53.0
Average		37.2						287	55.2
---excluding starred items								215	41.4

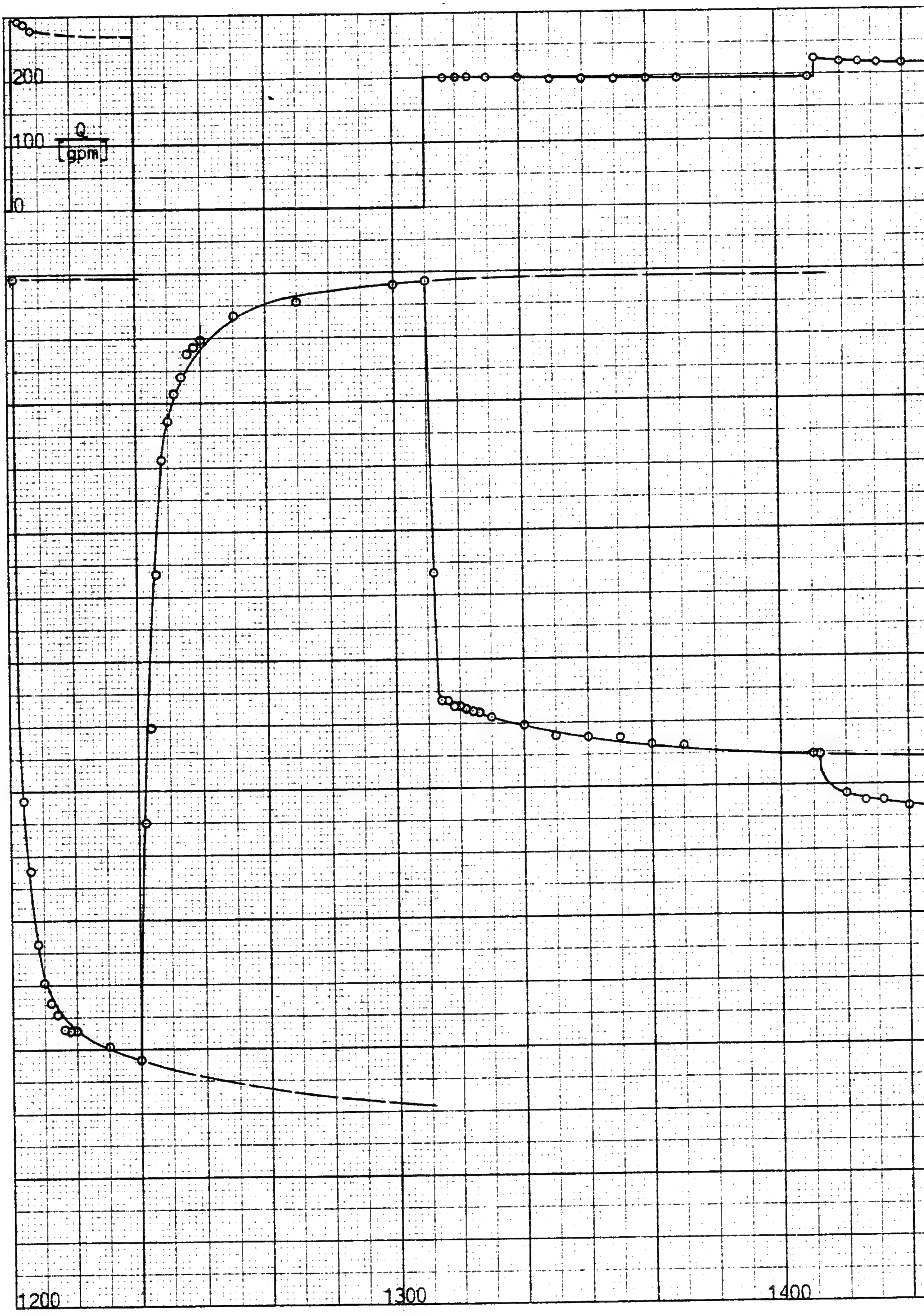
a) Depths shown to nearest foot are reported depths of drilling; depths shown to tenths or hundredths are sounded depths.

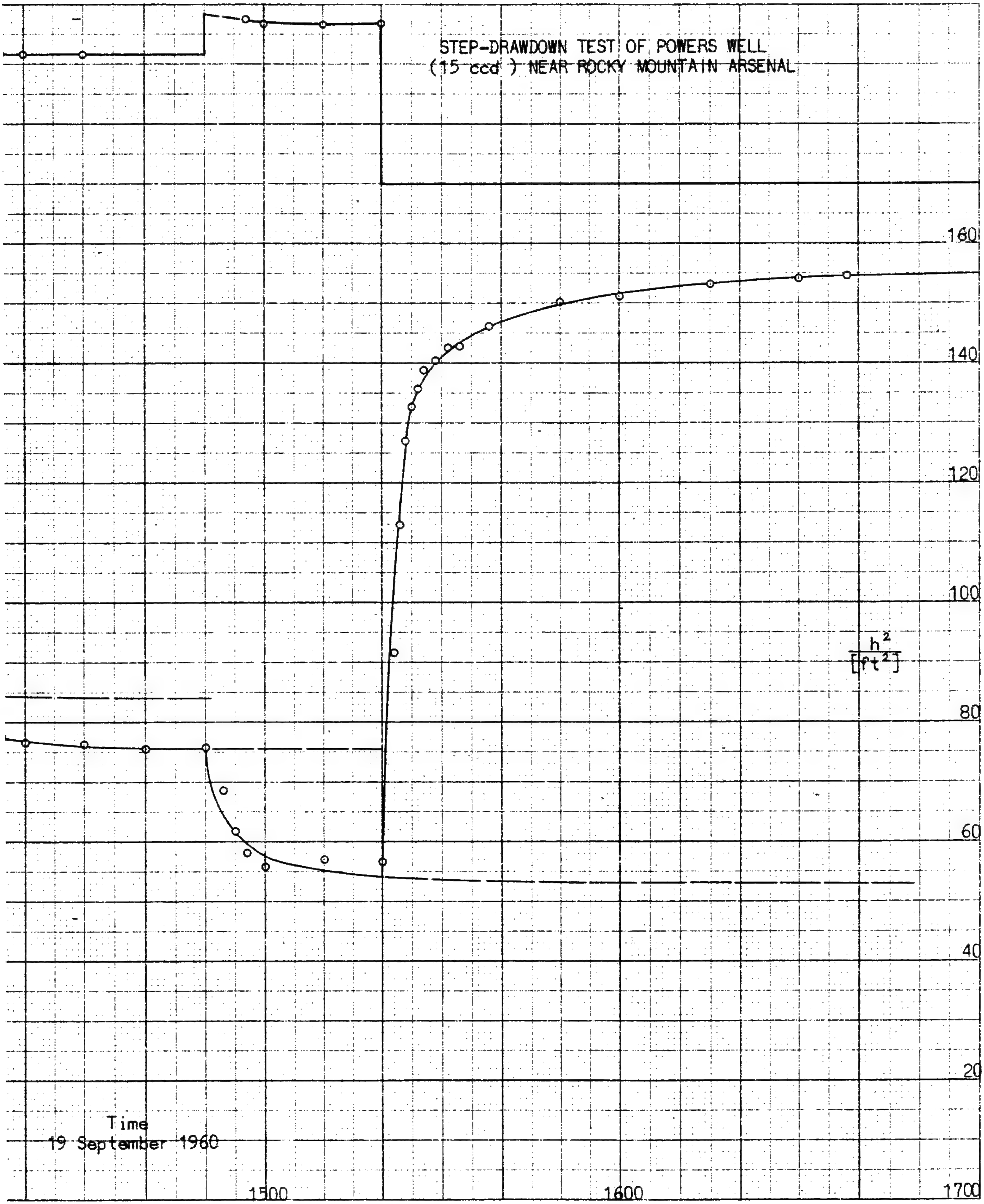
BLE D-1

TS OF WELLS NEAR ROCKY MOUNTAIN ARSENAL

[10] red arge, $^3ft^3/day$	[11] Drawdown over 1 log-cycle in time $\Delta_1(h^2)/[ft^2]$	[12] Hydraulic conductivity K/[ft/day]	[13] Drawdown at 100 min $\Delta(h^2)/[ft^2]$	[14] $\frac{\Delta(h^2)}{\Delta(h^2)}$	[15] Log $\frac{S^* r_w^2}{h^*[ft]}$	[16] Initial depth of flow, $h^0/[ft]$
37.4	10.3	1,330	86.1 - 29.0	5.55	0.77 - 4	9.3
8.1	2.5	1,190	131.8 - 113.5	7.33	.94 - 6	11.5
147.0*	18.	3,000*	377 - 76	16.7	.97 - 15*	19.4*
33.9	12.	1,040	102.8 - 39.8	5.25	.96 - 4	10.1
64.7	9.0	2,640	131.8 - 44.0	9.76	.85 - 8	11.5
74.0*	10.	2,720*	201.9 - 54.0	14.8	.83 - 13*	14.2*
74.4	12.5	2,180	148.9 - 76.0	5.83	.70 - 4	12.2
8.9	2.5-	1,300+	12.25 + .05	4.9+	.41 - 3	3.5
50.9	18.	1,040	118	6.55	.66 - 5	11.8
53.0	17.	1,140	150 - 18	7.75	.50 - 6	12.3
55.2		1,758			.96 - 7	11.6
41.4		1,483			.72 - 5	10.3

013-M2 601116 1/3 CEJ/epf




STEP-DRAWDOWN TEST OF POWERS WELL
(15 ccd) NEAR ROCKY MOUNTAIN ARSENAL
160
140
120
100
$\frac{h^2}{[ft^2]}$
80
60
40
20
Time
19 September 1960
1500
1600
1700

013-M2-601116 5/15 CEJ/epf

EUGENE DIETZGEN CO.

NO. 3400-20 DIETZGEN GRAPH PAPER 20 X 20 PER INCH

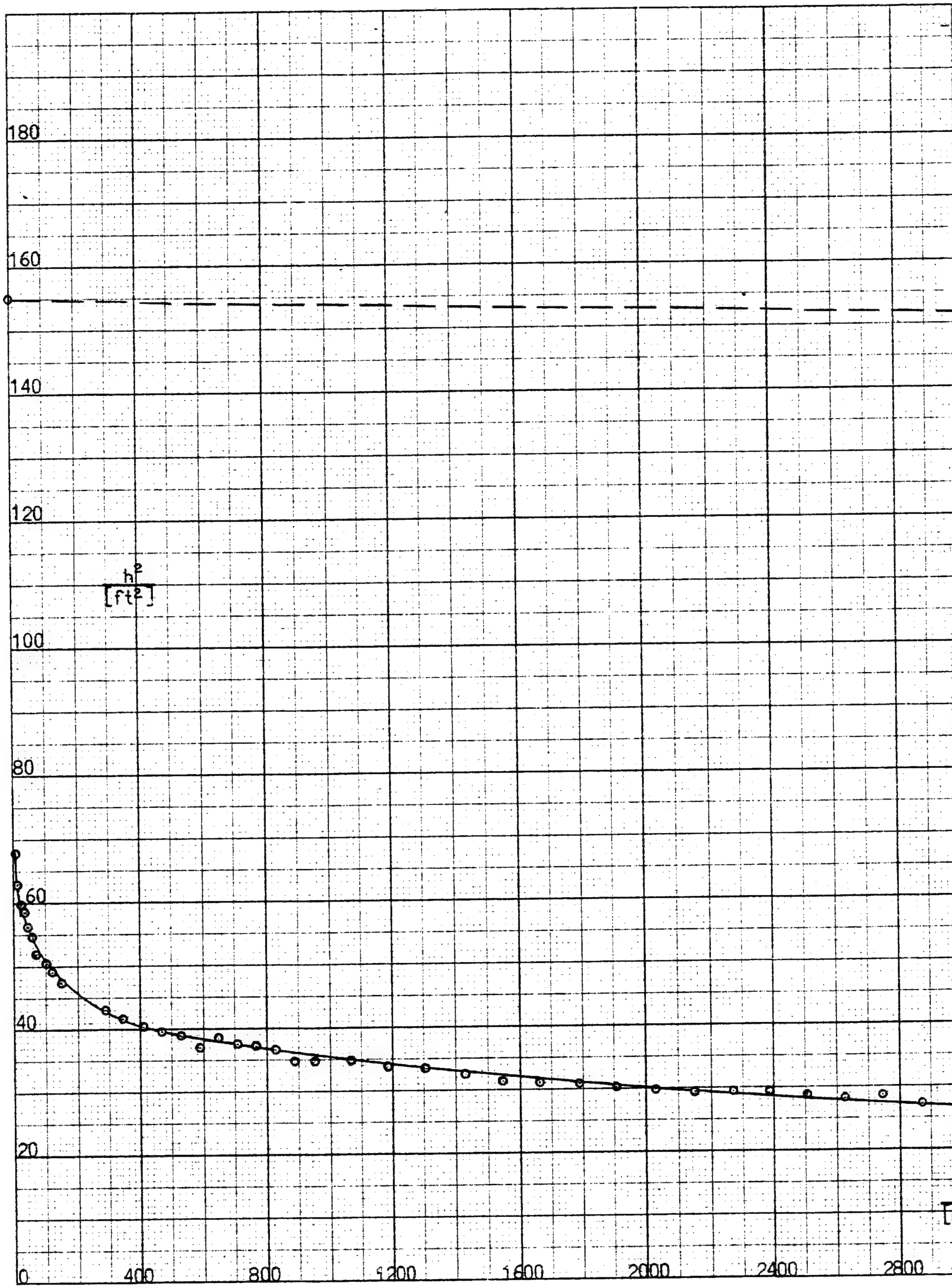
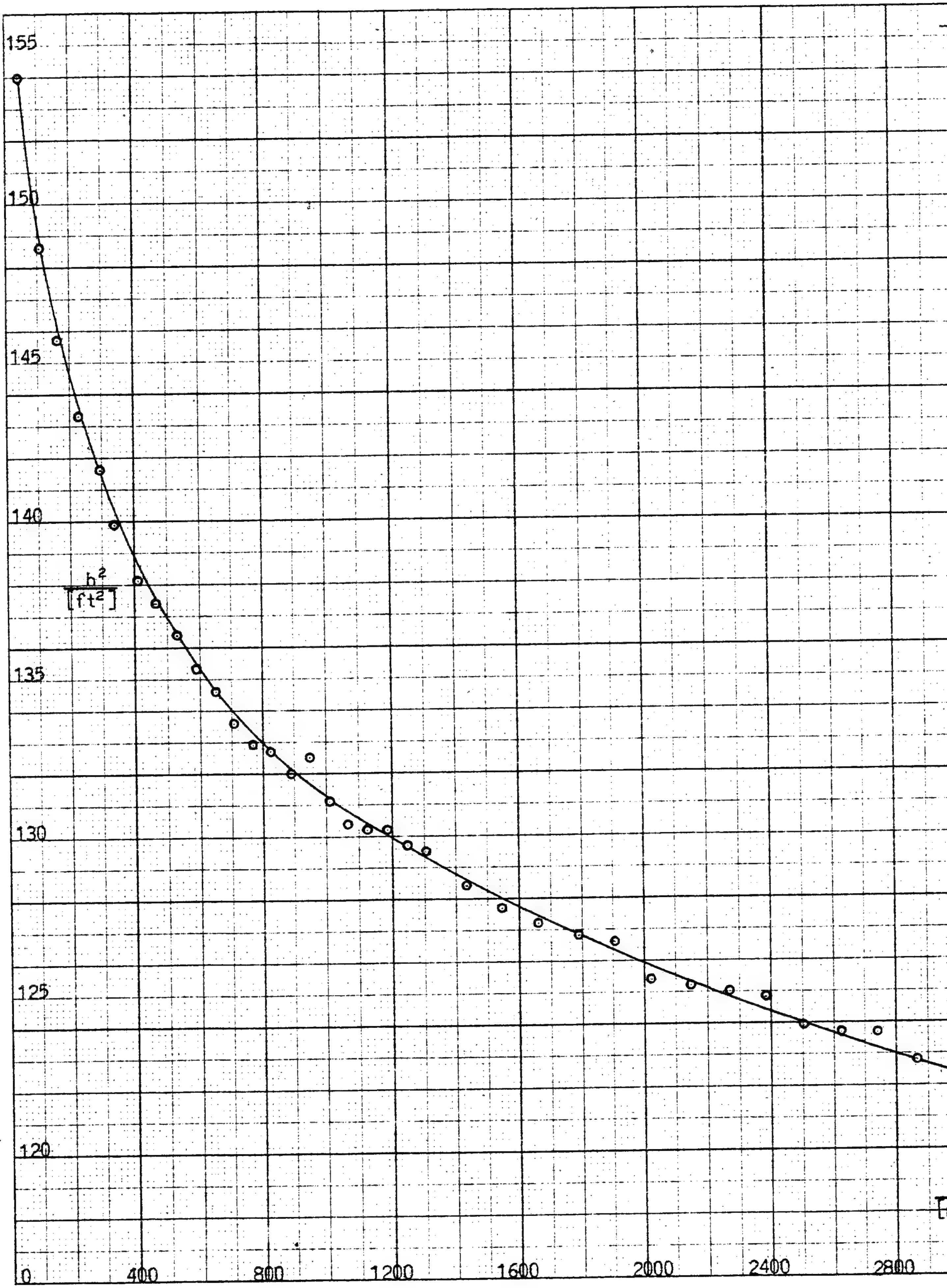


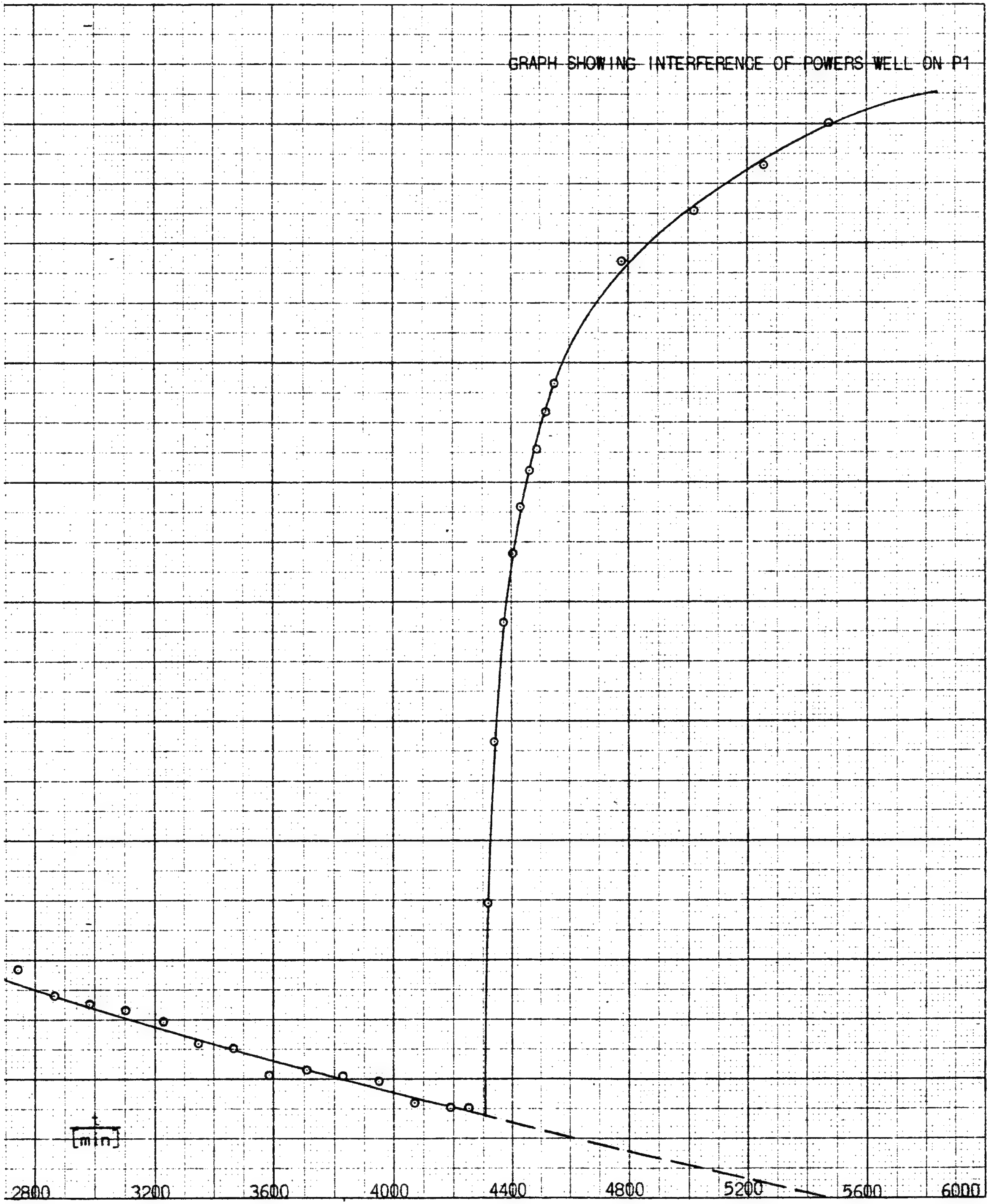
FIG. D-24


DRAWDOWN AND RECOVERY GRAPH OF POWERS WELL,
OCTOBER 9 TO 13, 1960
t
[min]
2800
3200
3600
4000
4400
4800
5200
5600
6000

013-M2-601116 5/15 CEJ/epf



F'G. D-25


GRAPH SHOWING INTERFERENCE OF POWERS WELL ON P1
t
[min]
2800
3200
3600
4000
4400
4800
5200
5600
6000

013-M2-601116 6/15 CEJ/epf

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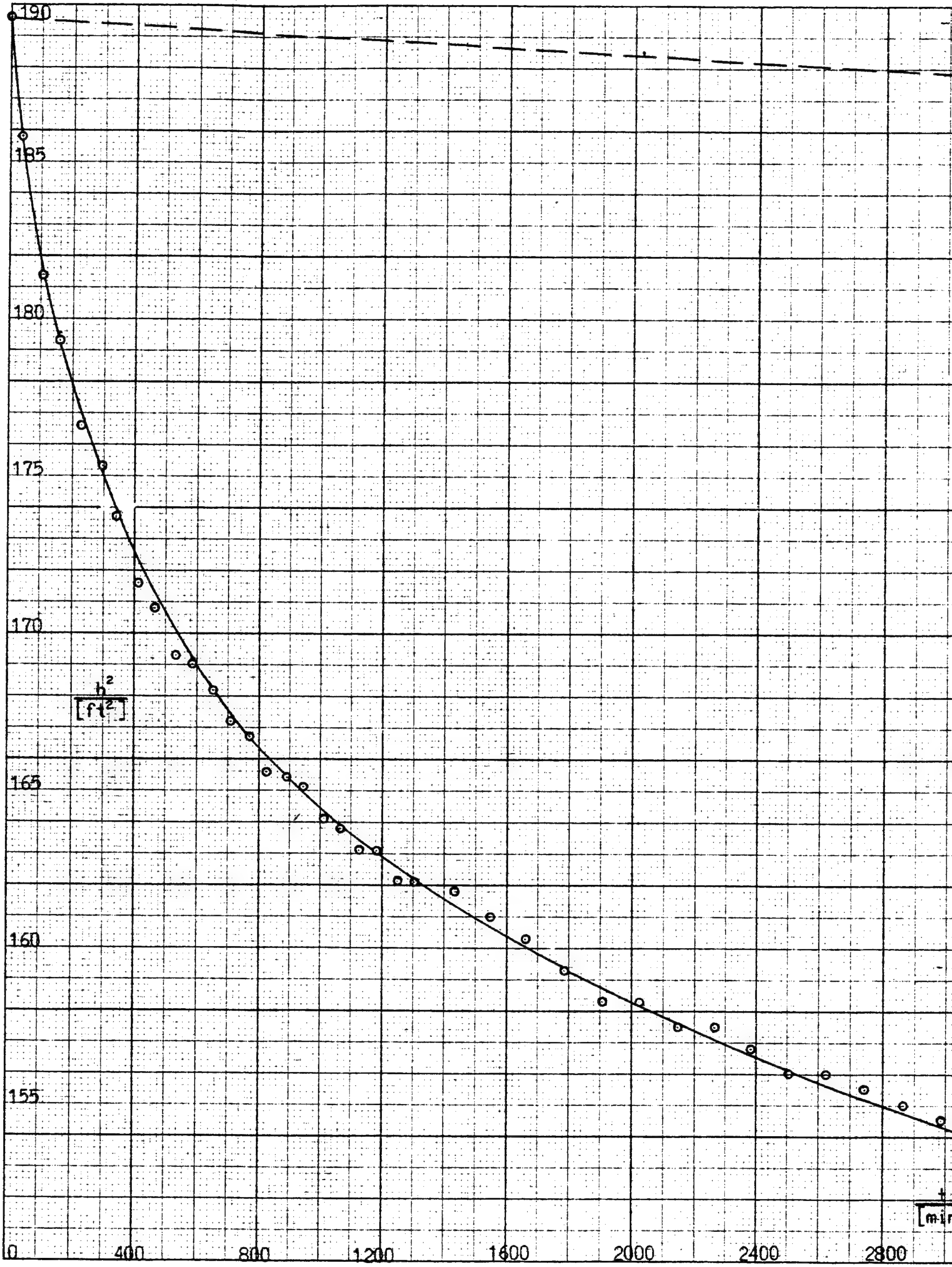
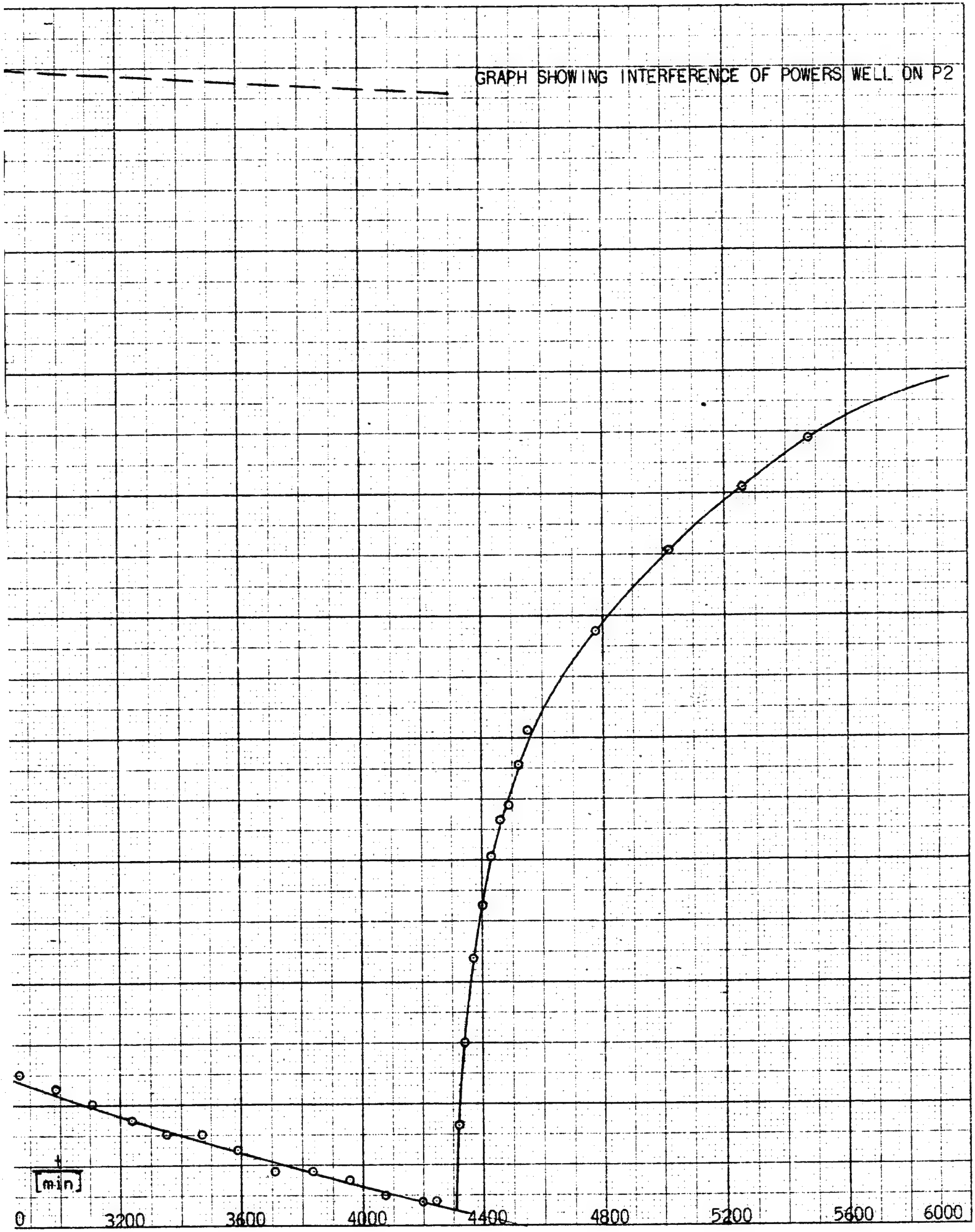
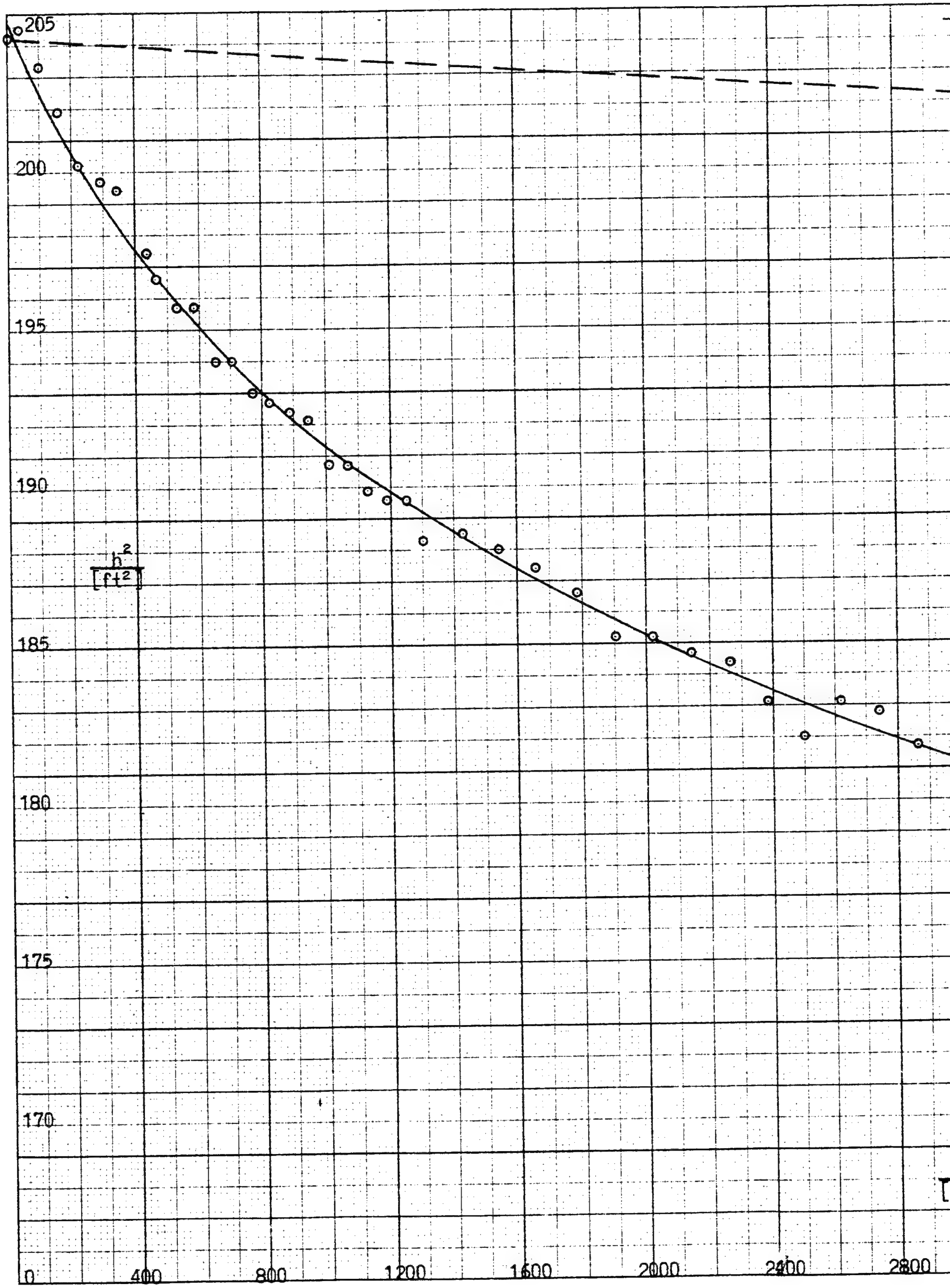
NO. 3400-20 DIETZGEN GRAPH PAPER 20 X 20 PER INCH



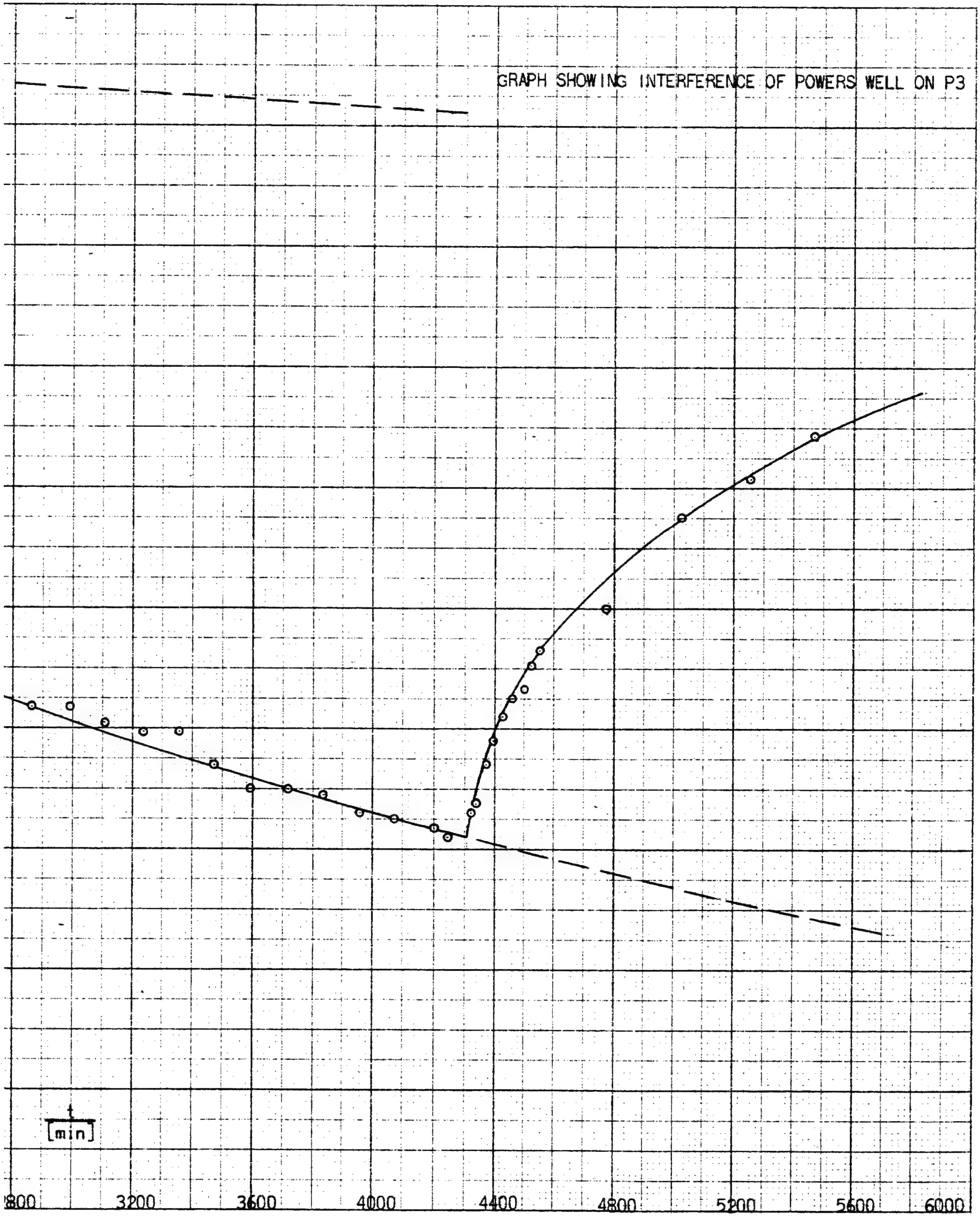

GRAPH SHOWING INTERFERENCE OF POWERS WELL ON P2
t
[min]
0
3200
3600
4000
4400
4800
5200
5600
6000

013-M2-601116 7/15 CEJ/epf

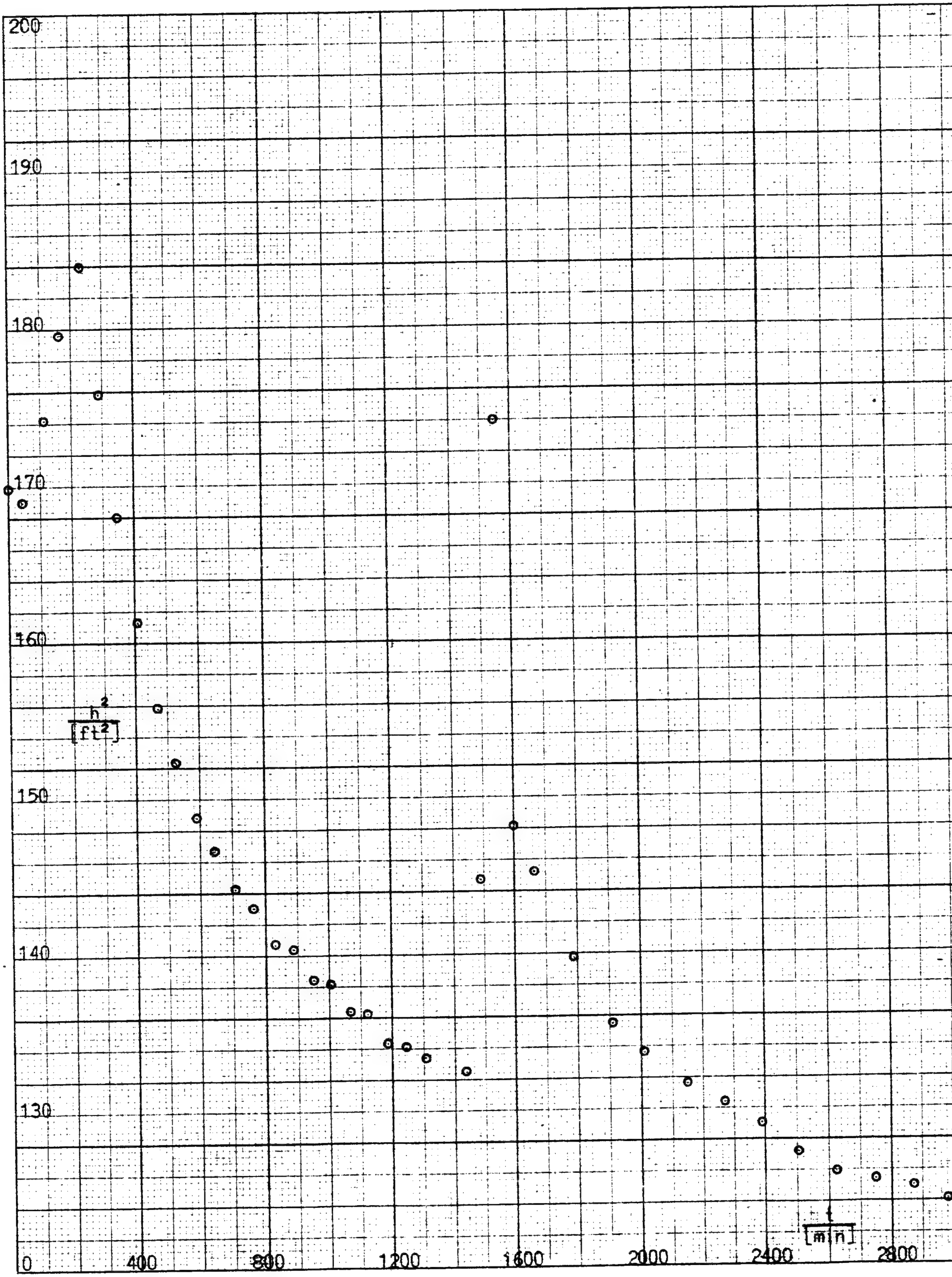
EUGENE DIETZGEN CO.
MADE IN U.S.A.

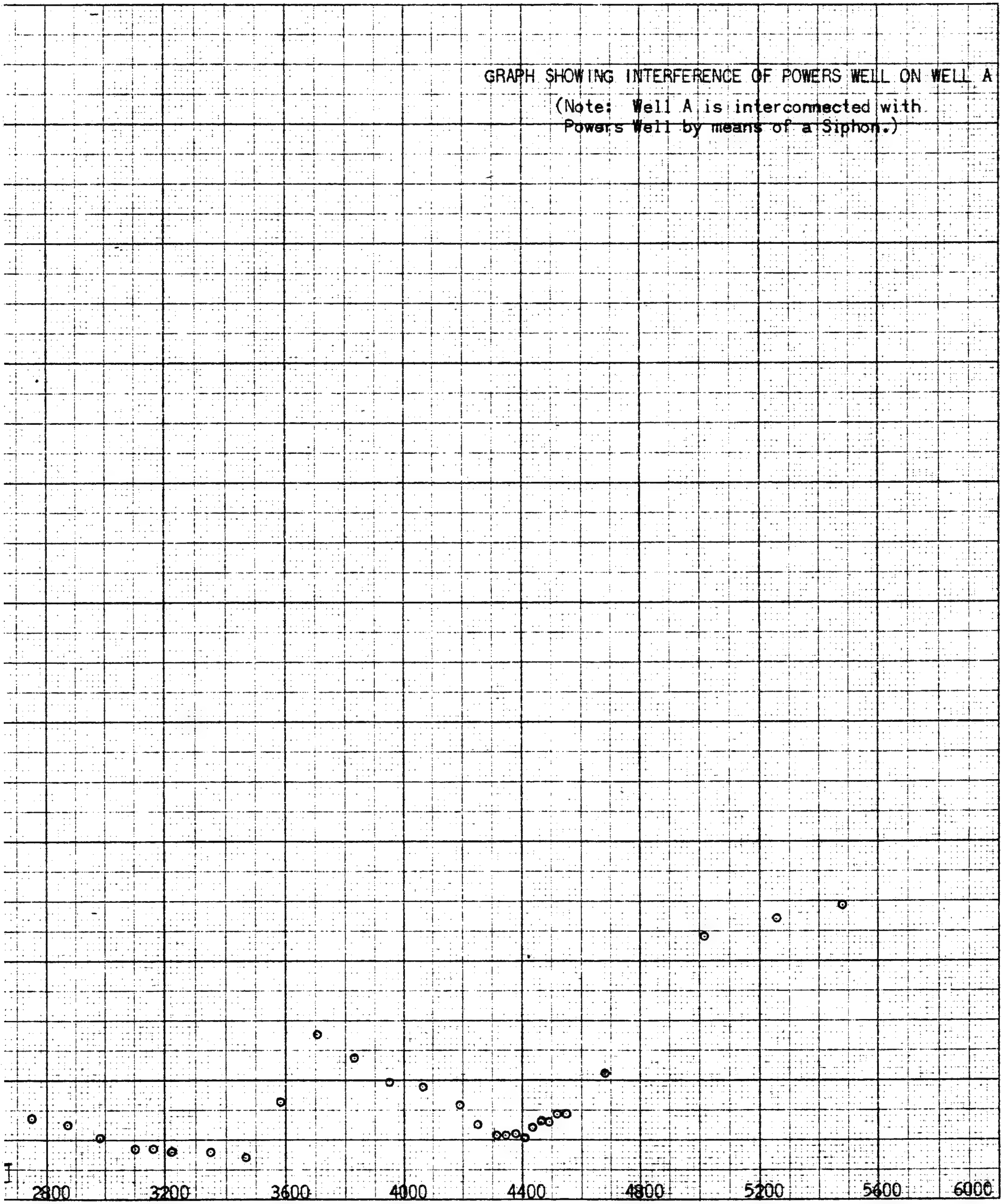


NO. 340D-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

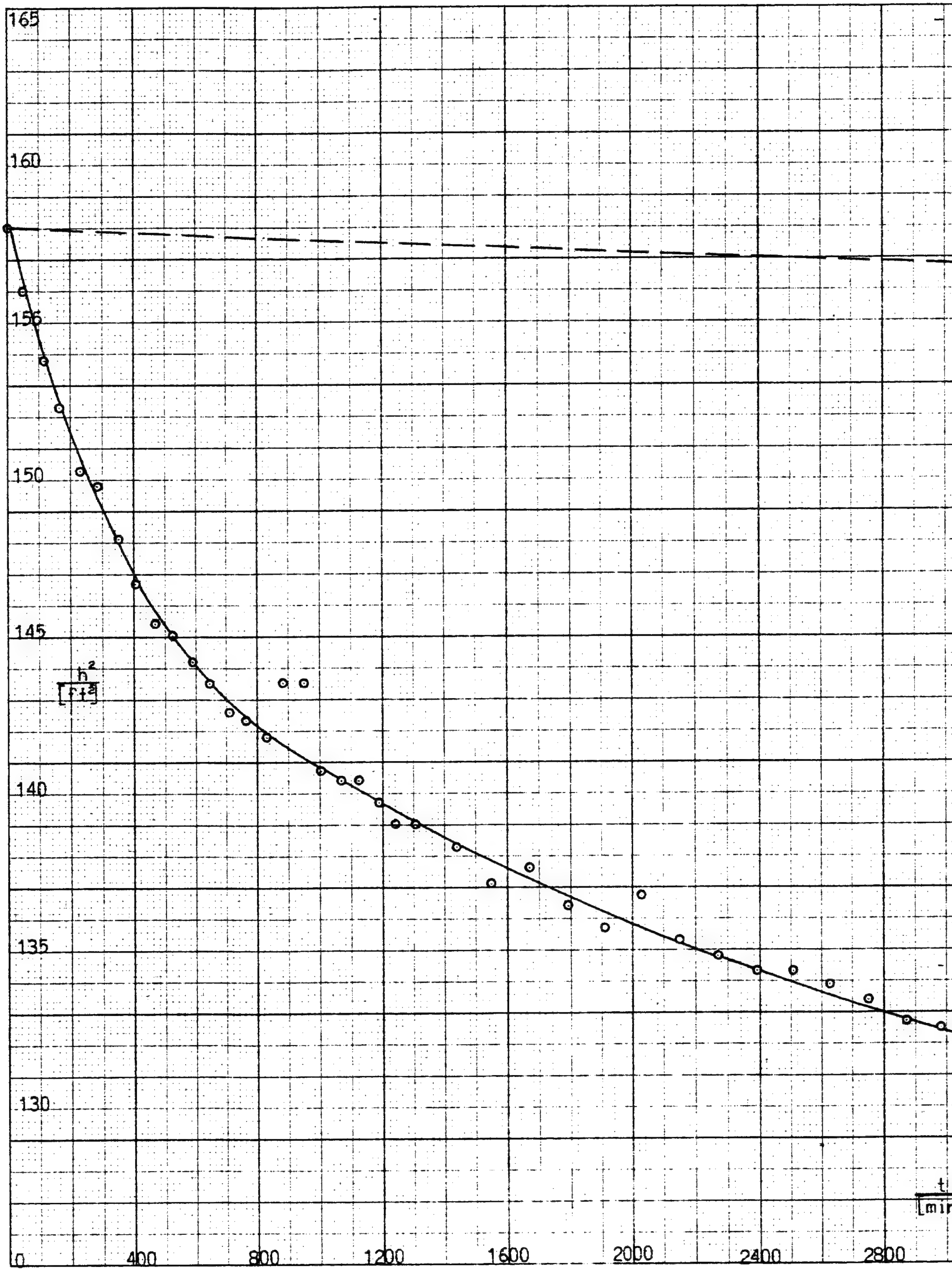

GRAPH SHOWING INTERFERENCE OF POWERS WELL ON P3
t
[min]
2800
3200
3600
4000
4400
4800
5200
5600
6000

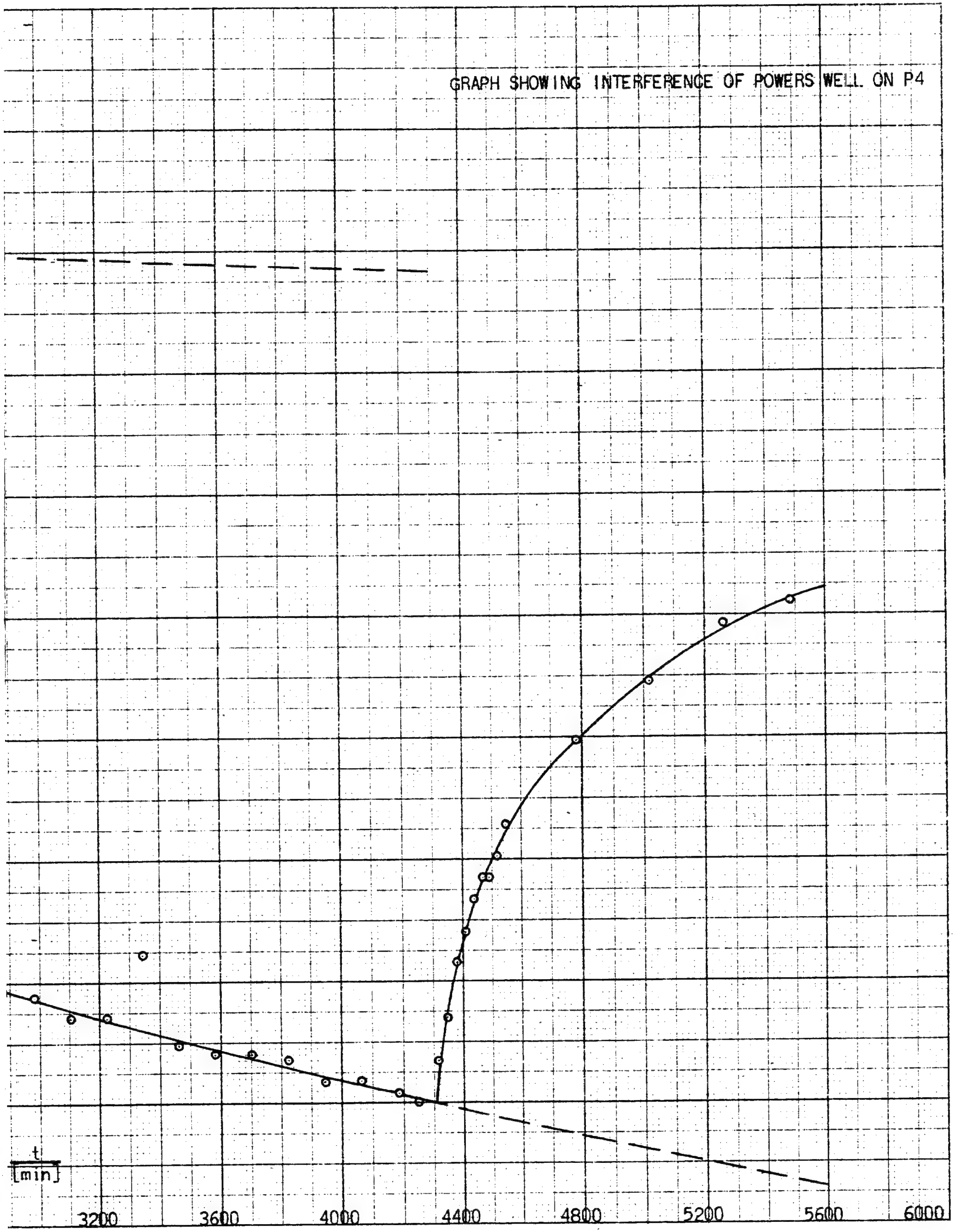
013-M2-601116 8/15 CEJ/epf




GRAPH SHOWING INTERFERENCE OF POWERS WELL ON WELL A
(Note: Well A is interconnected with Powers Well by means of a Siphon.)
2800
3200
3600
4000
4400
4800
5200
5600
6000

013-M2-601116 9/15 CEJ/epf




GRAPH SHOWING INTERFERENCE OF POWERS WELL ON P4
t
[min]
3200
3600
4000
4400
4800
5200
5600
6000

013-M2-601116 10/15 CEJ/epf

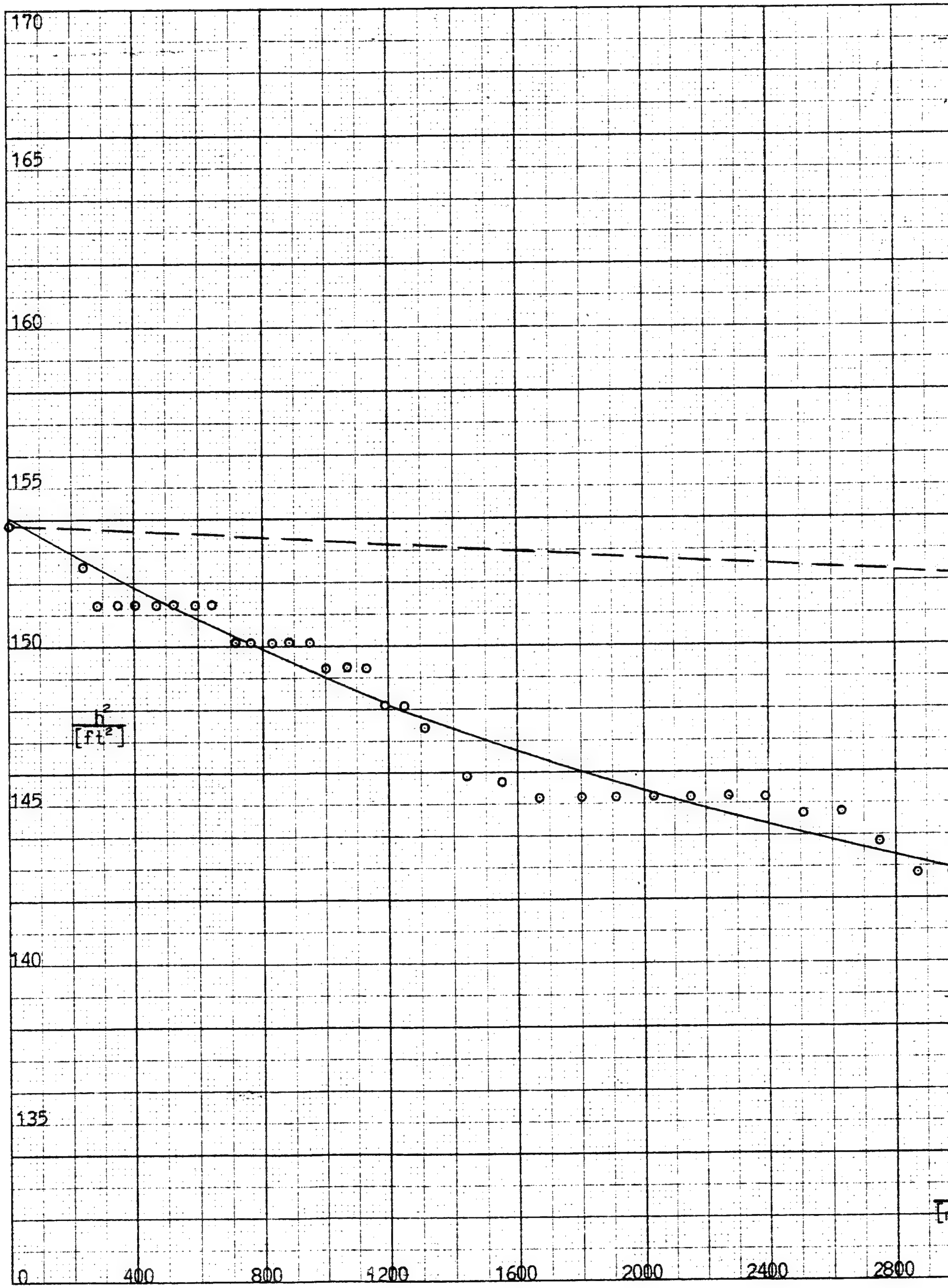
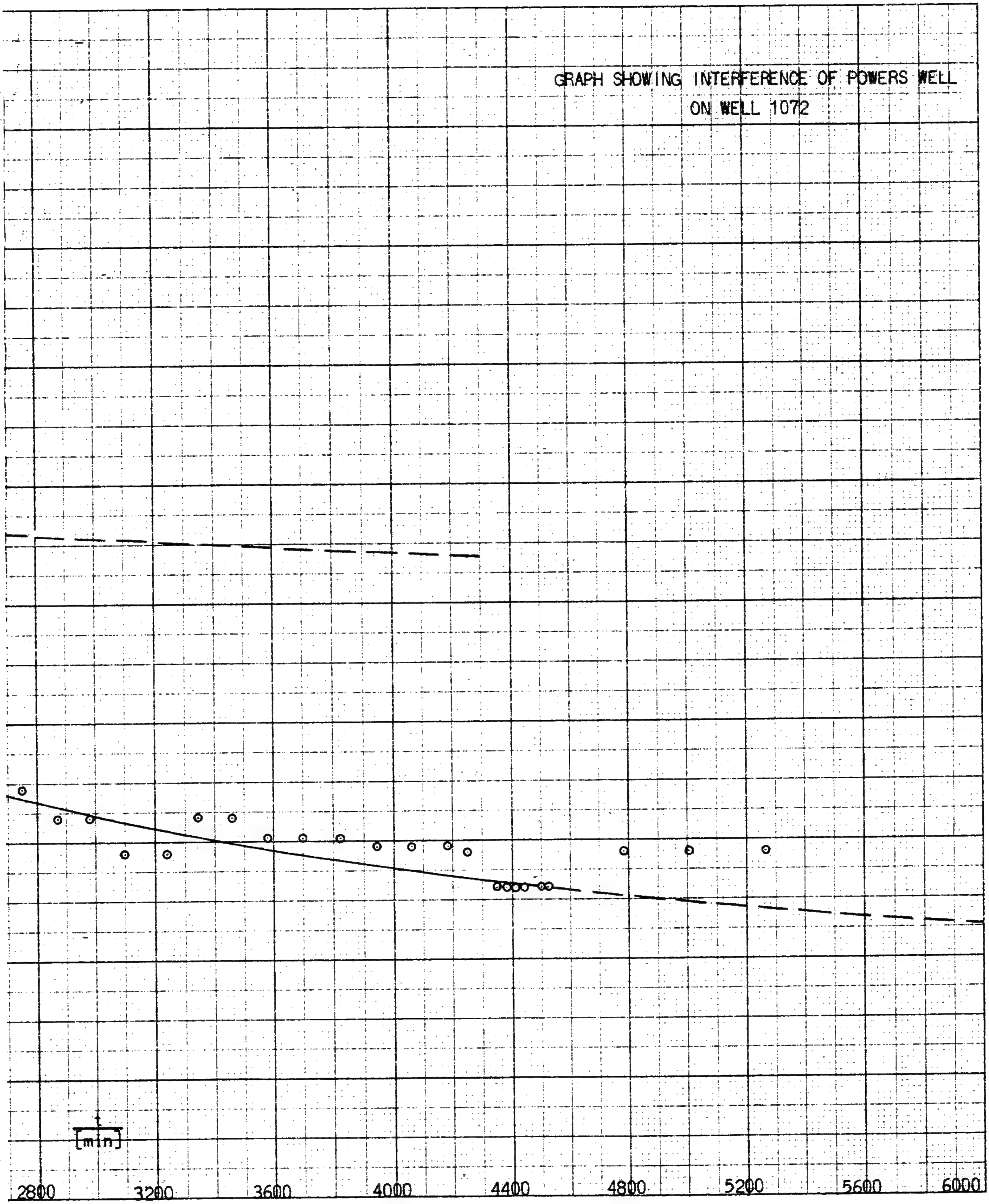


FIG. D-30


GRAPH SHOWING INTERFERENCE OF POWERS WELL ON WELL 1072
t [min]
2800
3200
3600
4000
4400
4800
5200
5600
6000

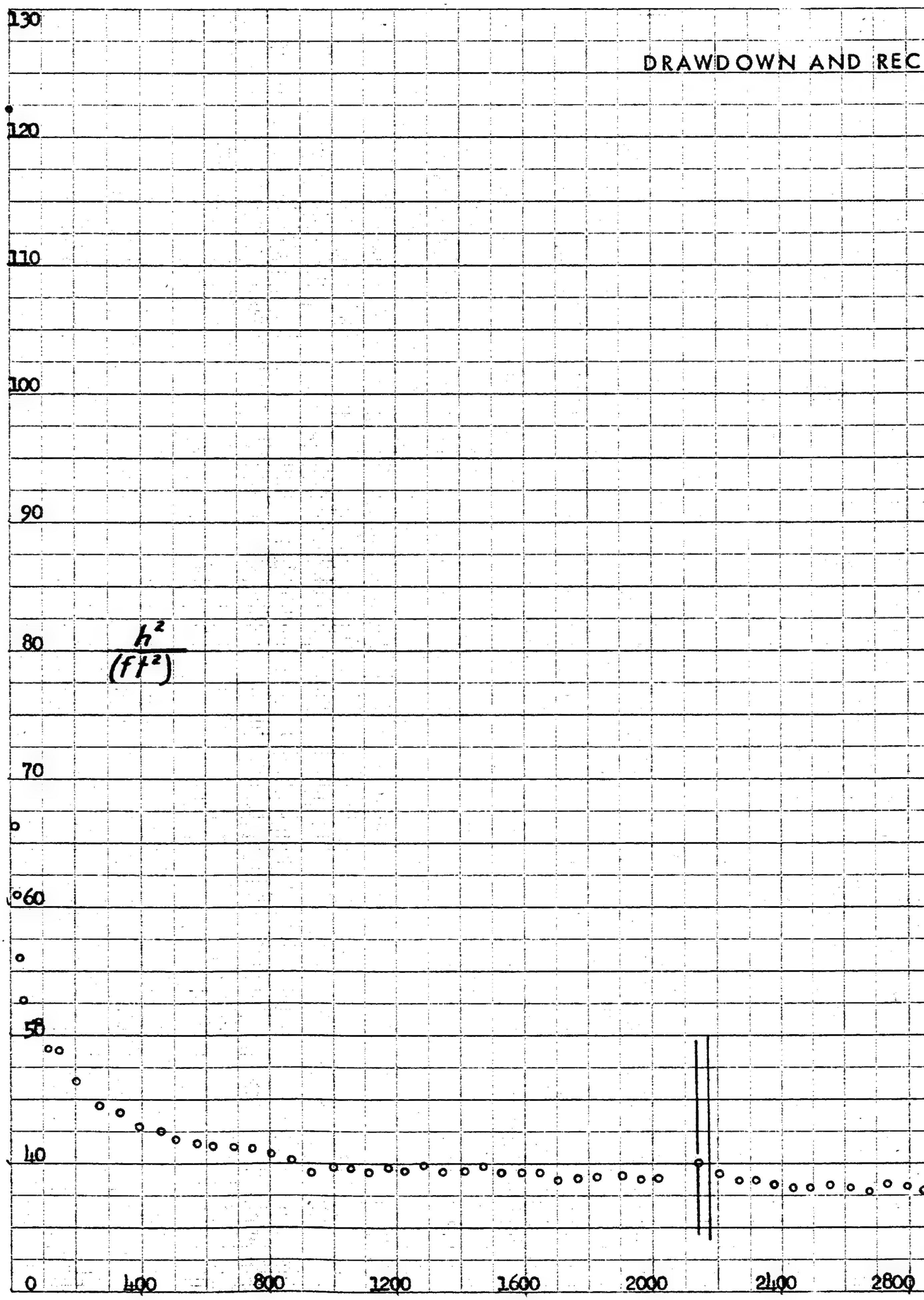
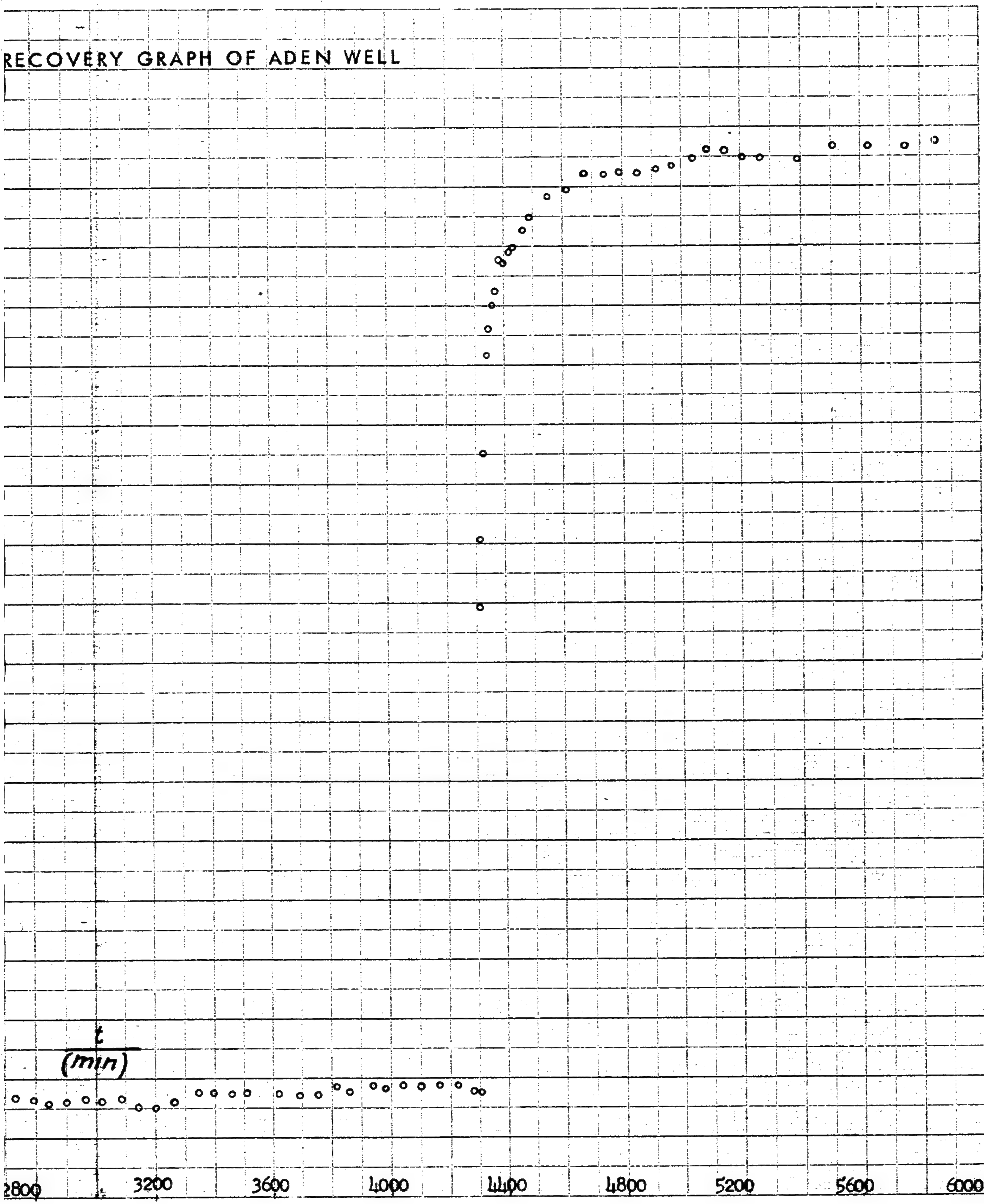
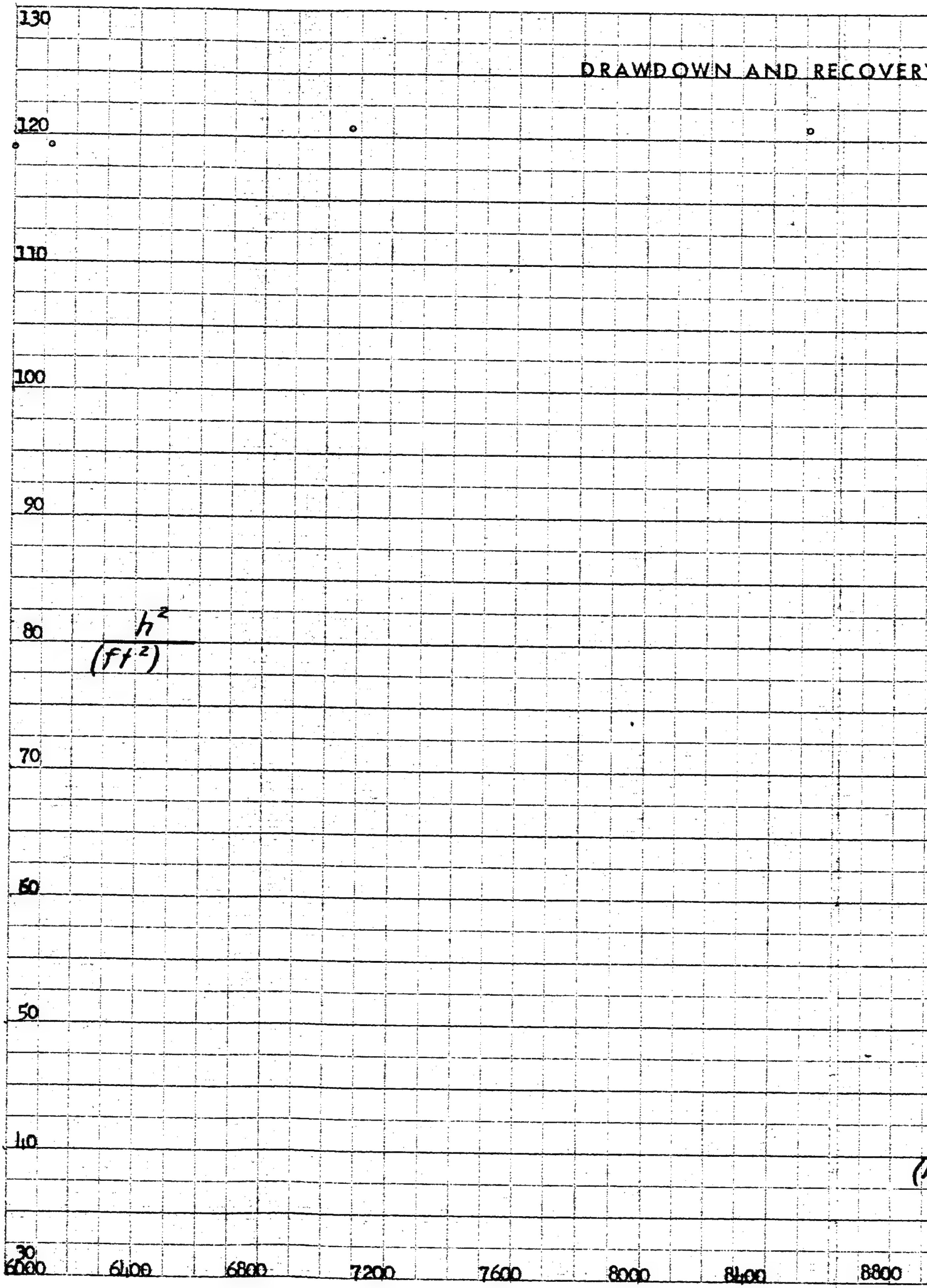
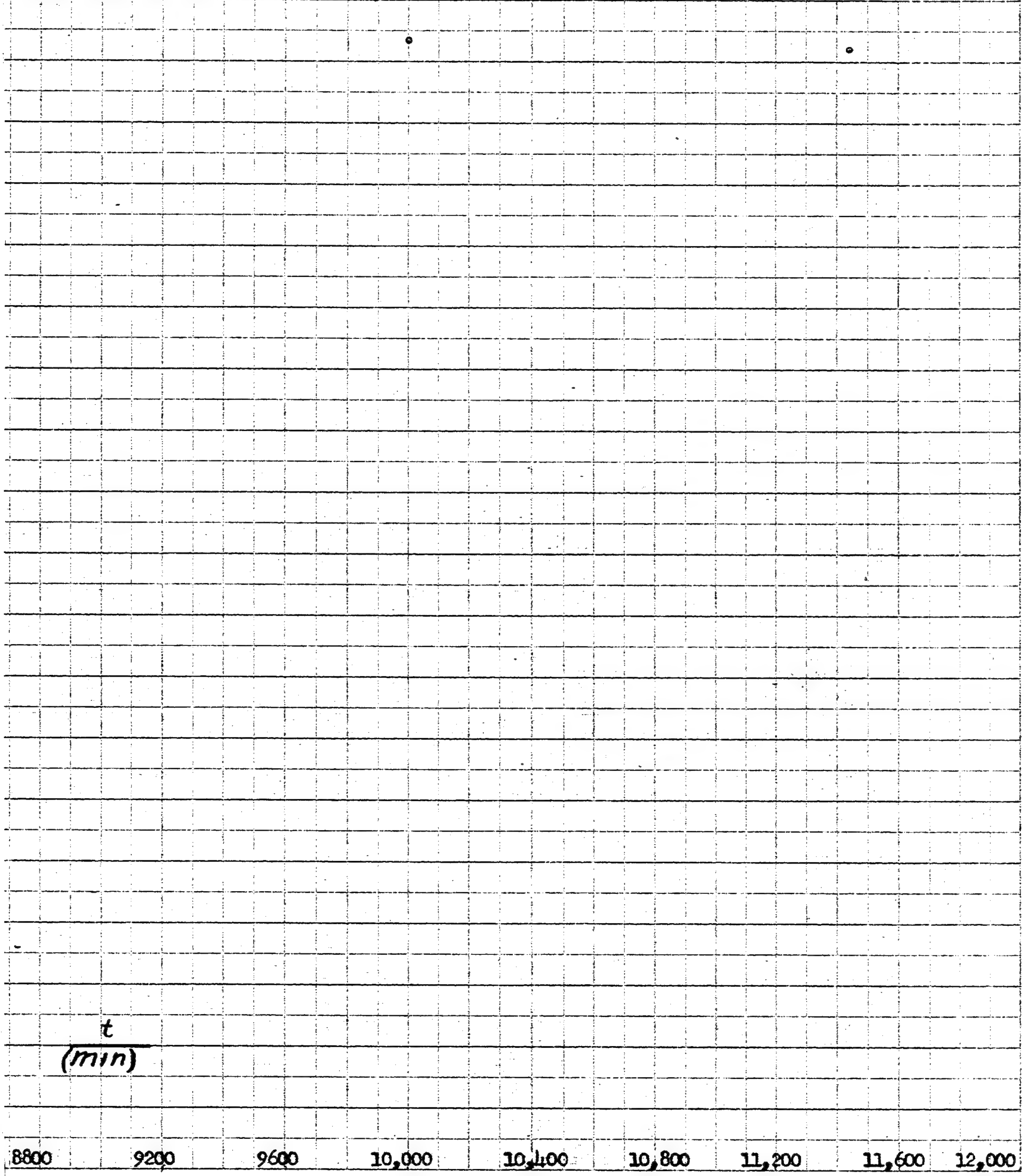

DRAWDOWN AND REC
130
120
110
100
90
80
70
60
50
40
$\frac{h^2}{(ft^2)}$
0
400
800
1200
1600
2000
2400
2800

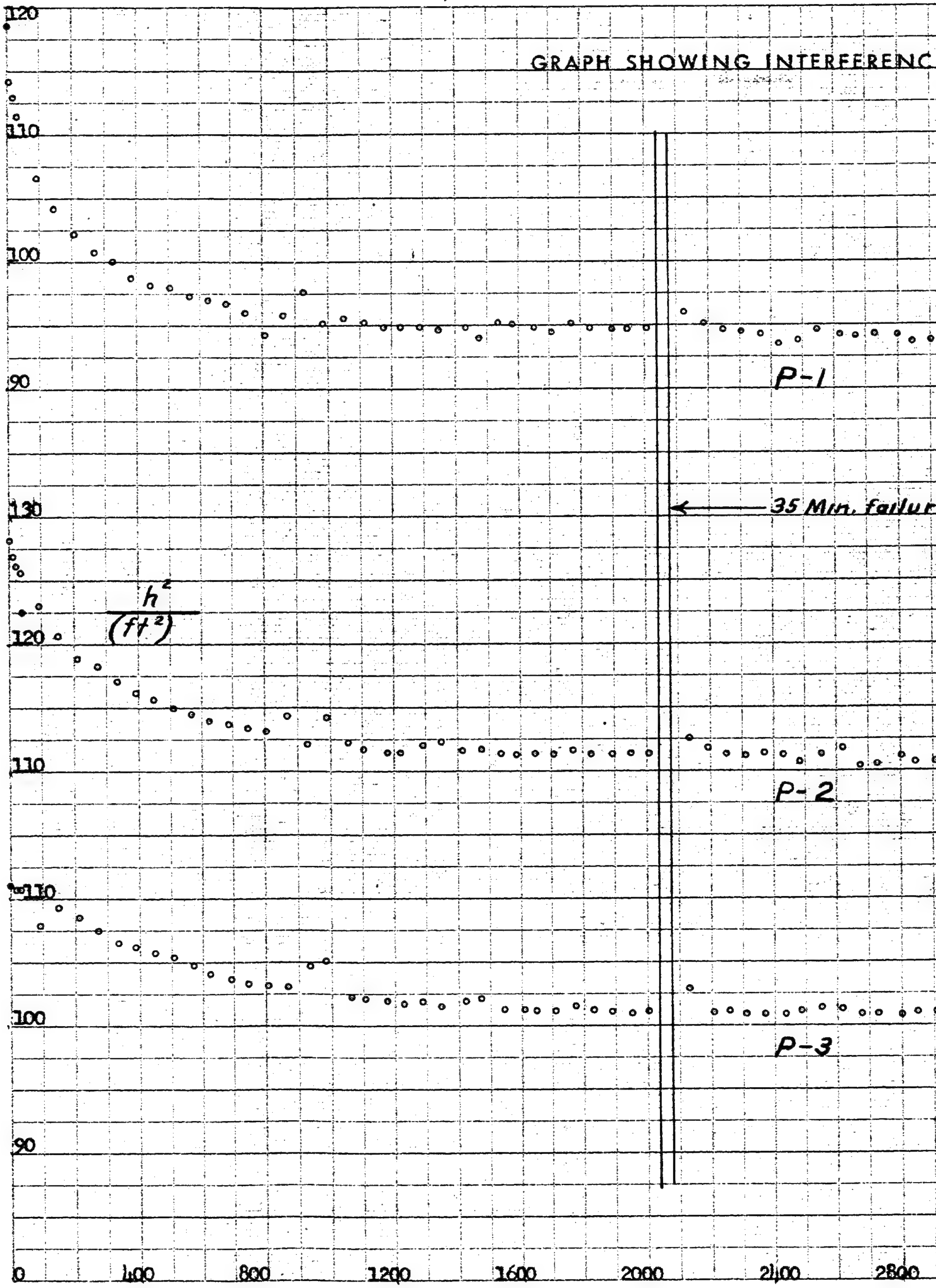
FIG. D-36a

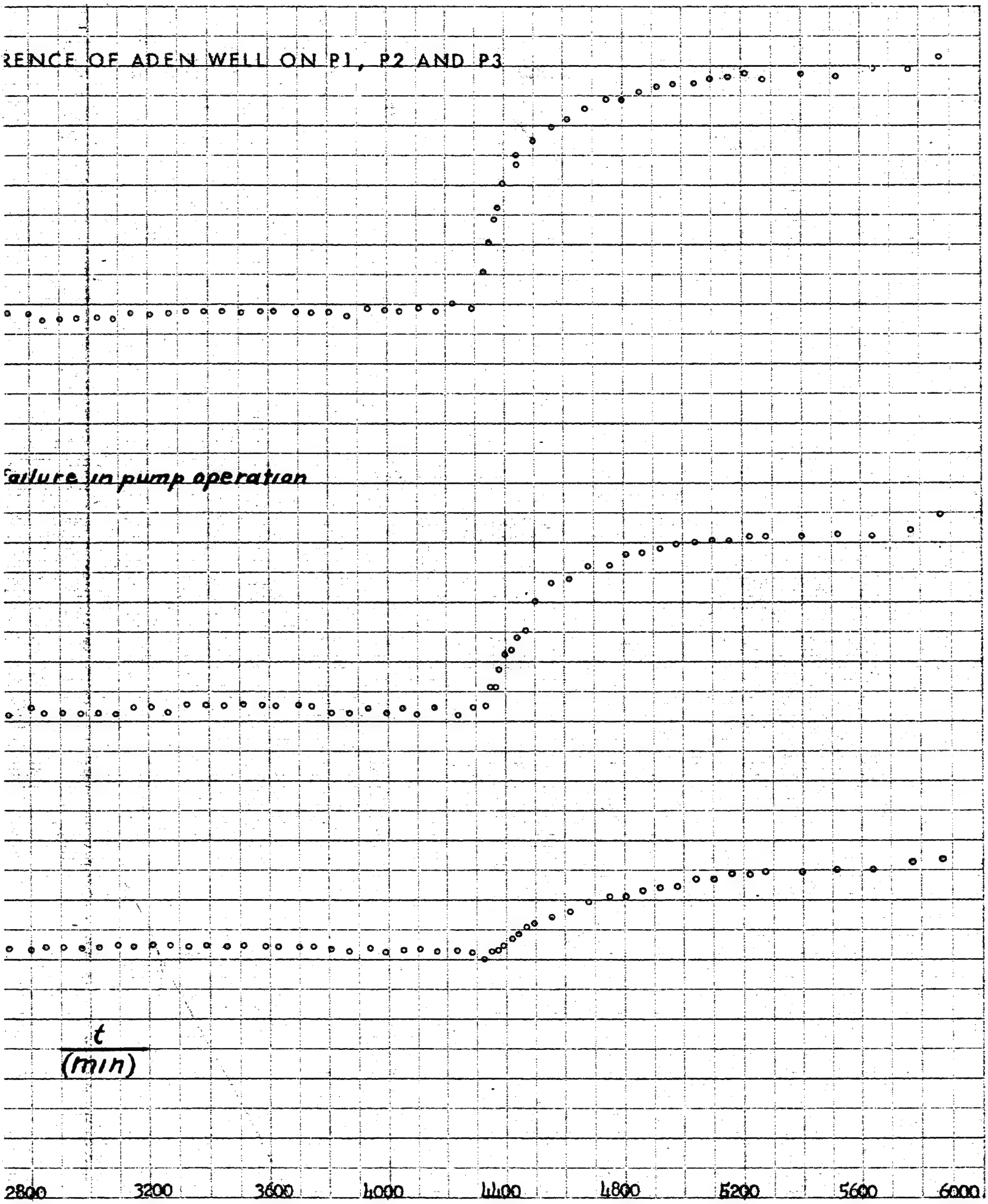

RECOVERY GRAPH OF ADEN WELL
t
(min)
2800
3200
3600
4000
4400
4800
5200
5600
6000


DRAWDOWN AND RECOVERY
130
120
110
100
90
80
h^2 (ft^2)
70
60
50
40
30
6000
6400
6800
7200
7600
8000
8400
8800

OVERY GRAPH OF ADEN WELL, Contd.




GRAPH SHOWING INTERFERENC
P-1
P-2
P-3
35 Min. failur
$\frac{h^2}{(ft^2)}$
120
110
100
90
130
120
110
110
100
90
0
400
800
1200
1600
2000
2400
2800


RENCE OF ADEN WELL ON P1, P2 AND P3
Failure in pump operation
t
(min)
2800
3200
3600
4000
4400
4800
5200
5600
6000

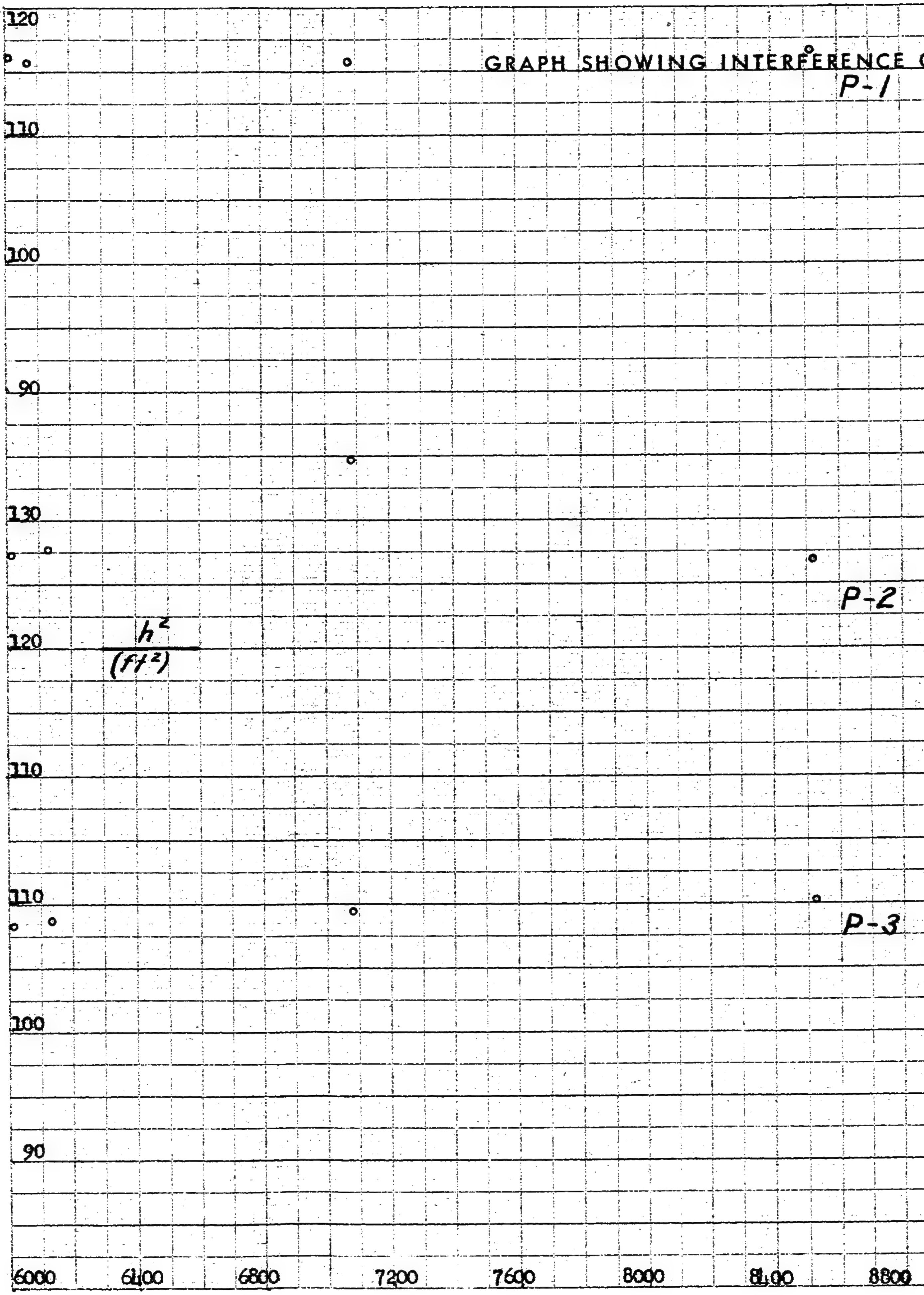
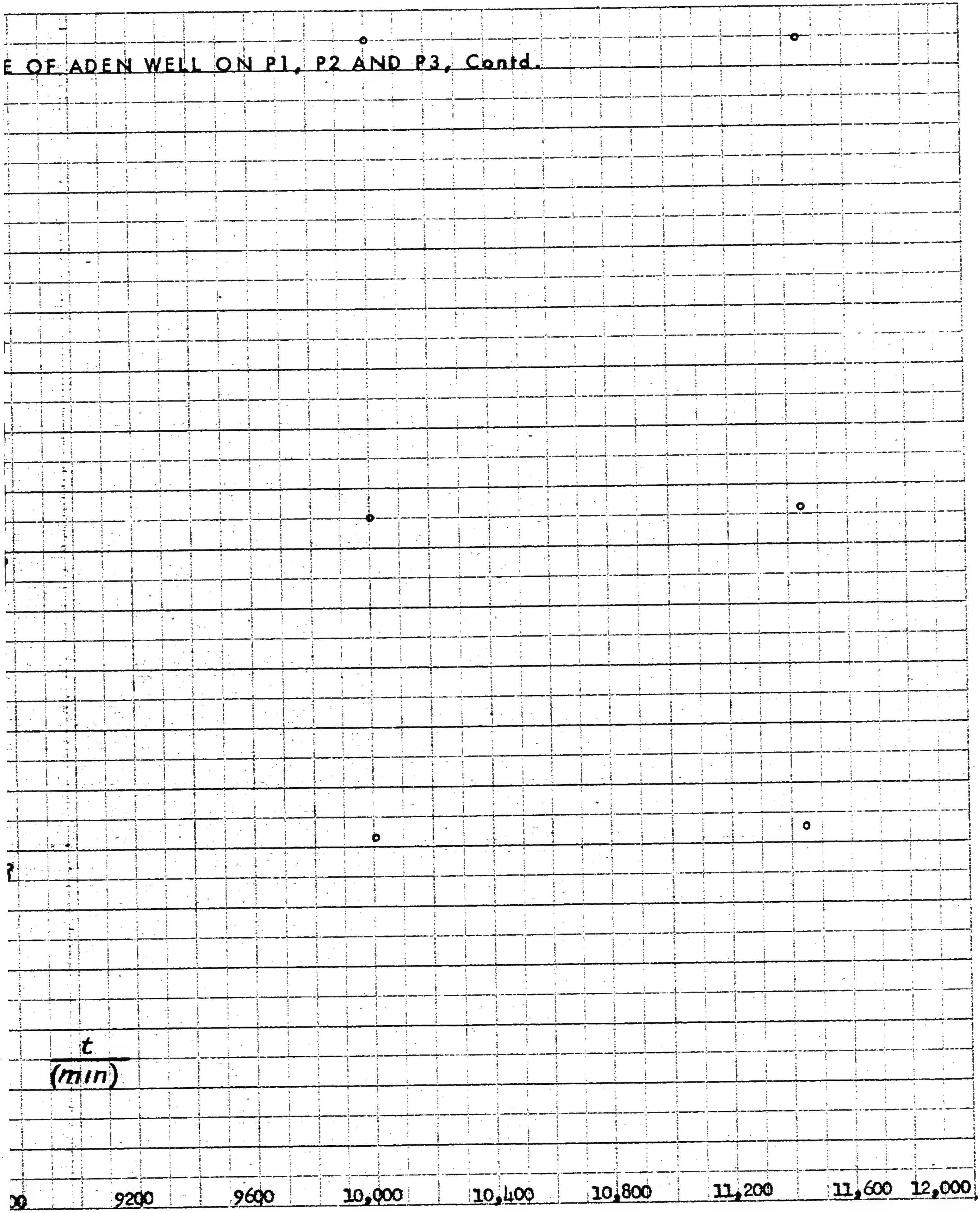
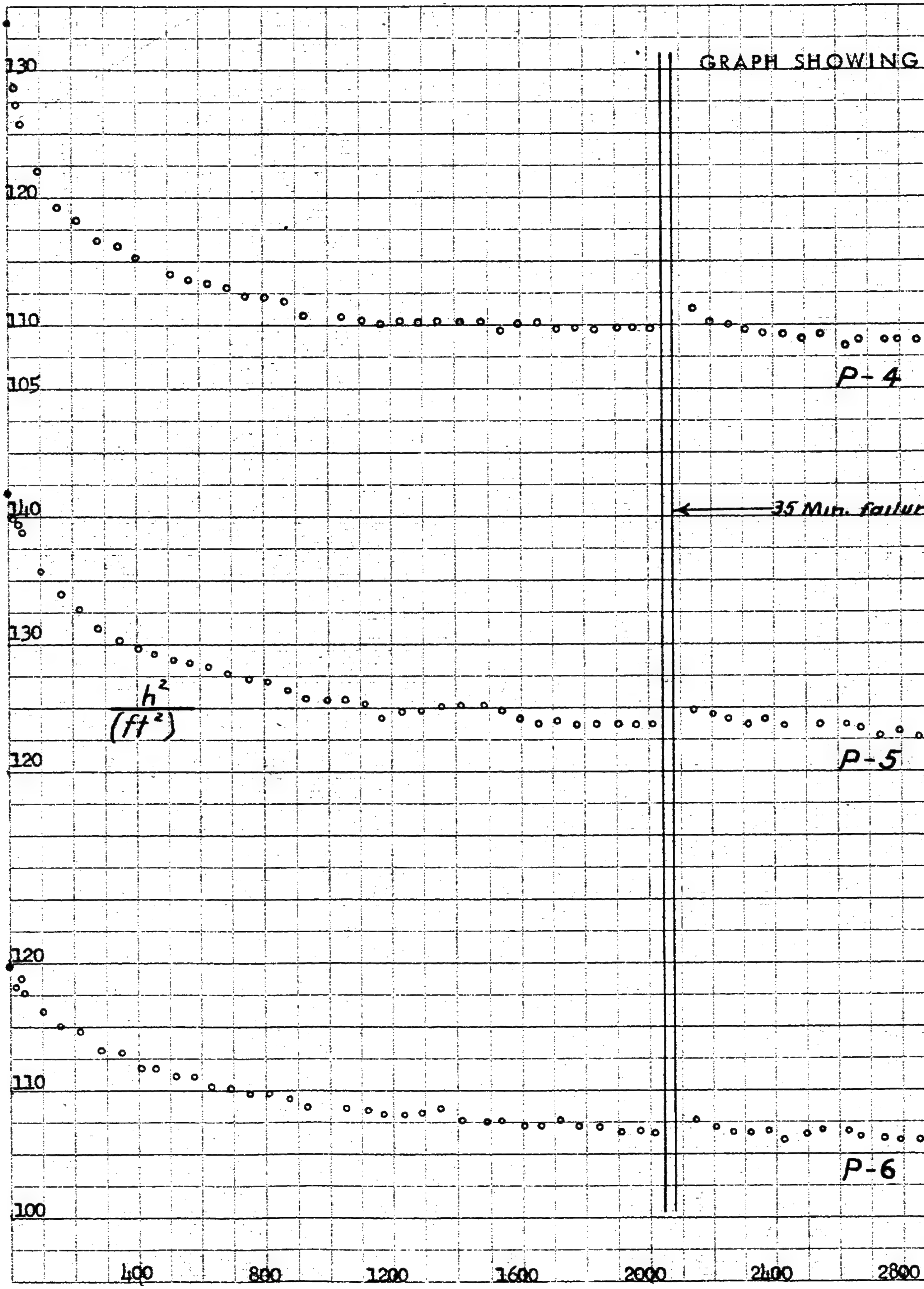
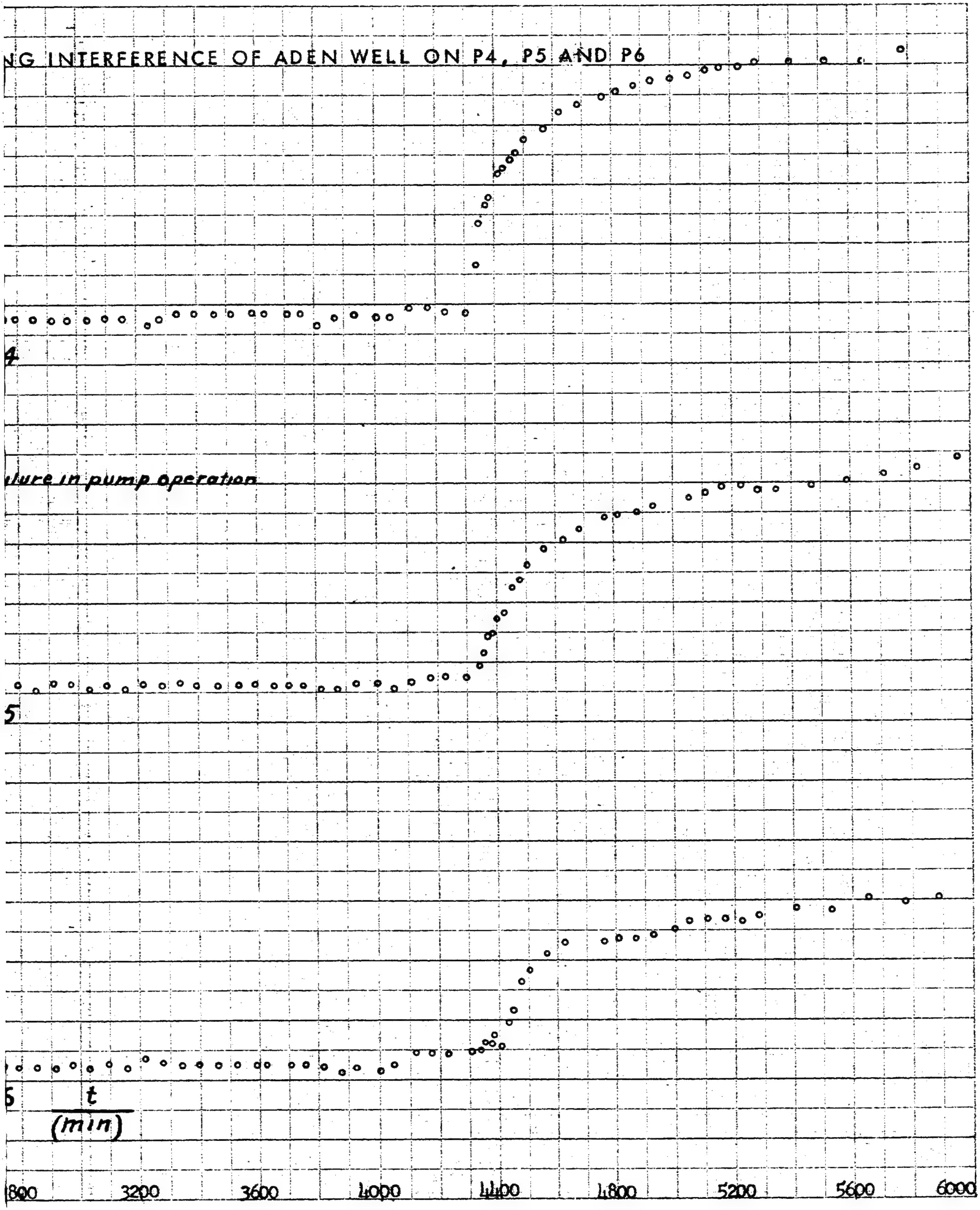

GRAPH SHOWING INTERFERENCE C
P-1
P-2
P-3
h^2 (ft^2)
120
110
100
90
130
120
110
110
100
90
6000
6400
6800
7200
7600
8000
8400
8800

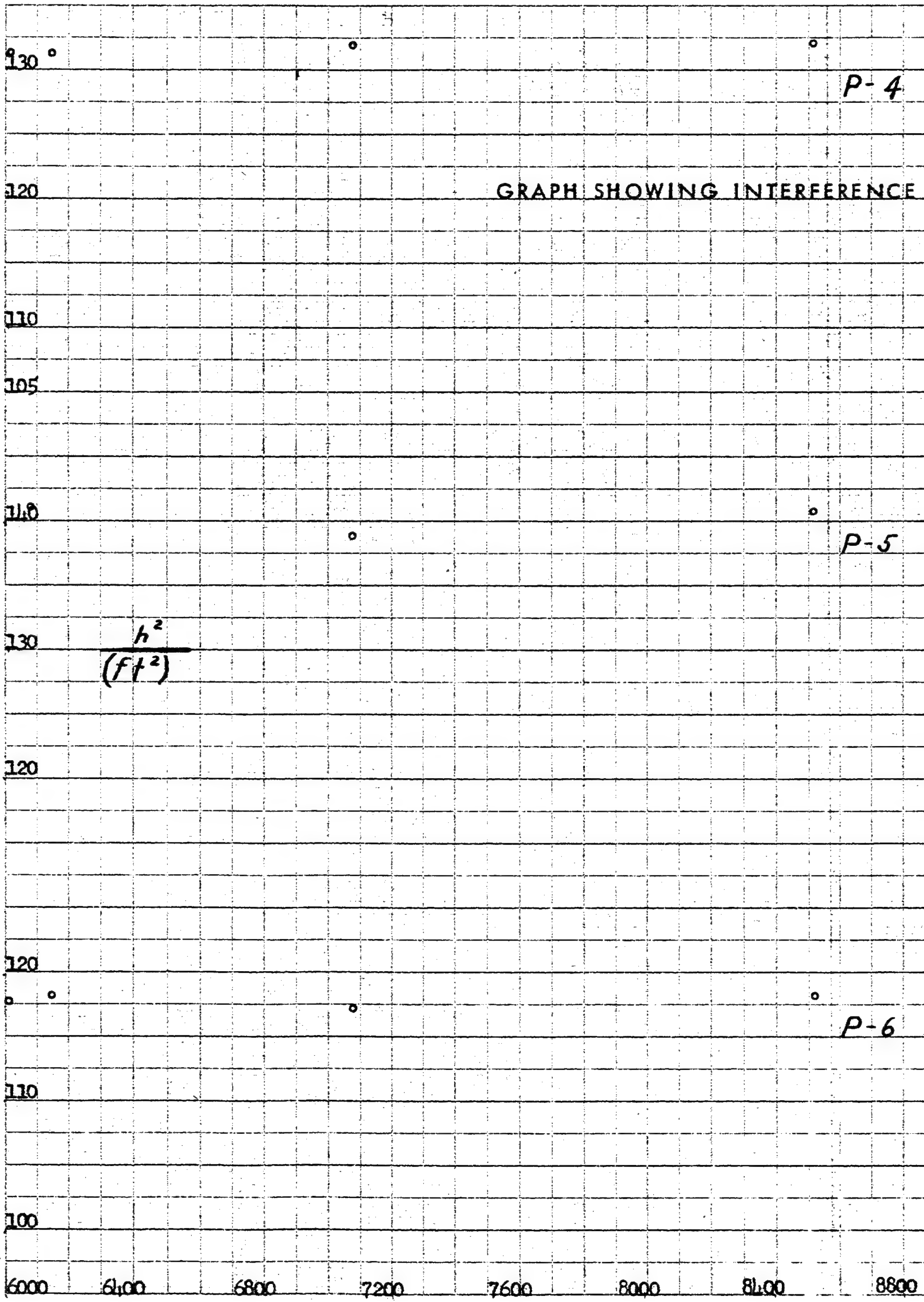
FIG. D-37b



E OF ADEN WELL ON P1, P2 AND P3, Contd.
t
(min)
00
9200
9600
10,000
10,400
10,800
11,200
11,600
12,000

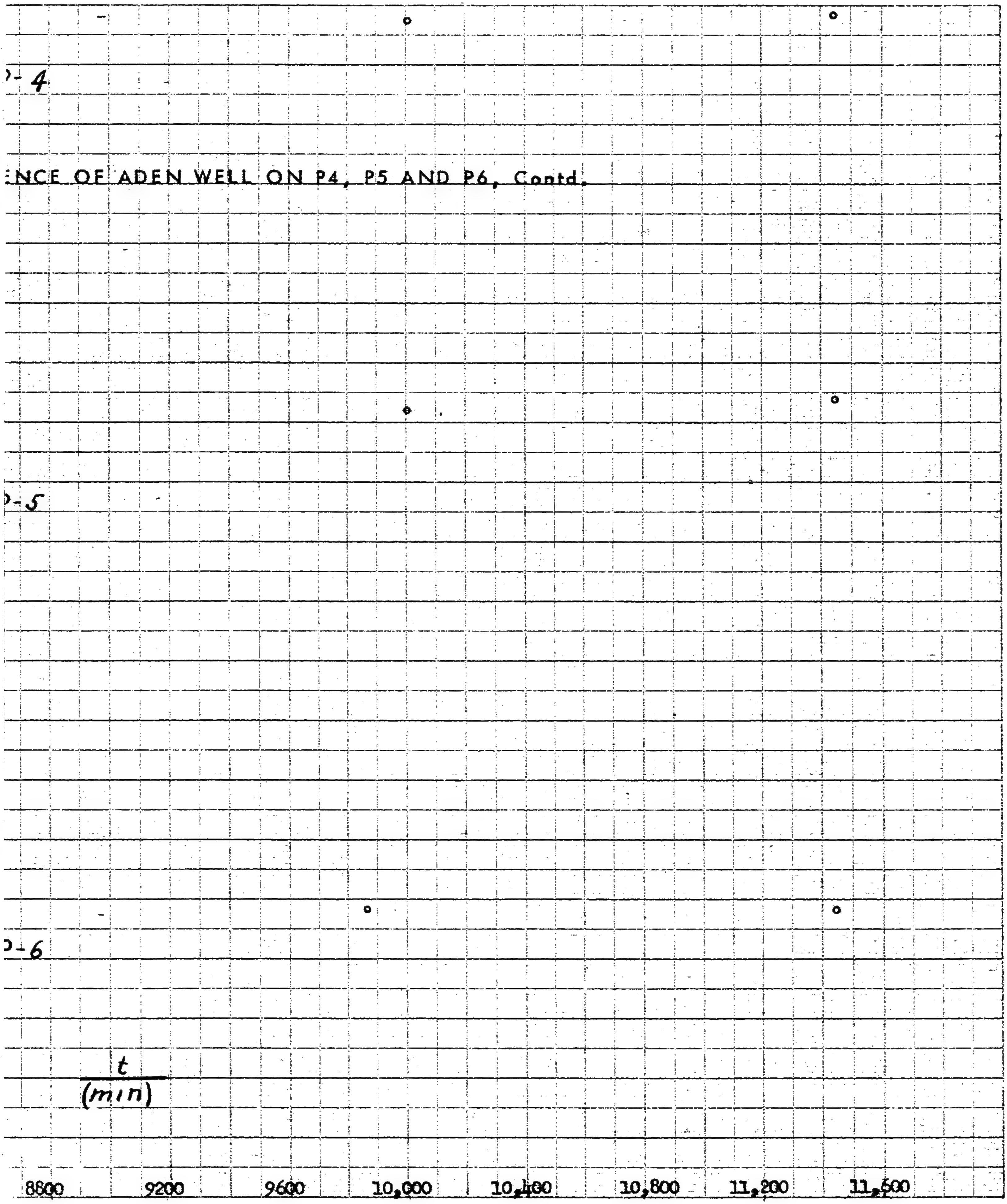

GRAPH SHOWING
130
120
110
105
P- 4
140
35 Min. failur
130
$\frac{h^2}{(ft^2)}$
120
P-5
120
110
P-6
100
400
800
1200
1600
2000
2400
2800


NG INTERFERENCE OF ADEN WELL ON P4, P5 AND P6
4
ilure in pump operation
5
6
t
(min)
2800
3200
3600
4000
4400
4800
5200
5600
6000

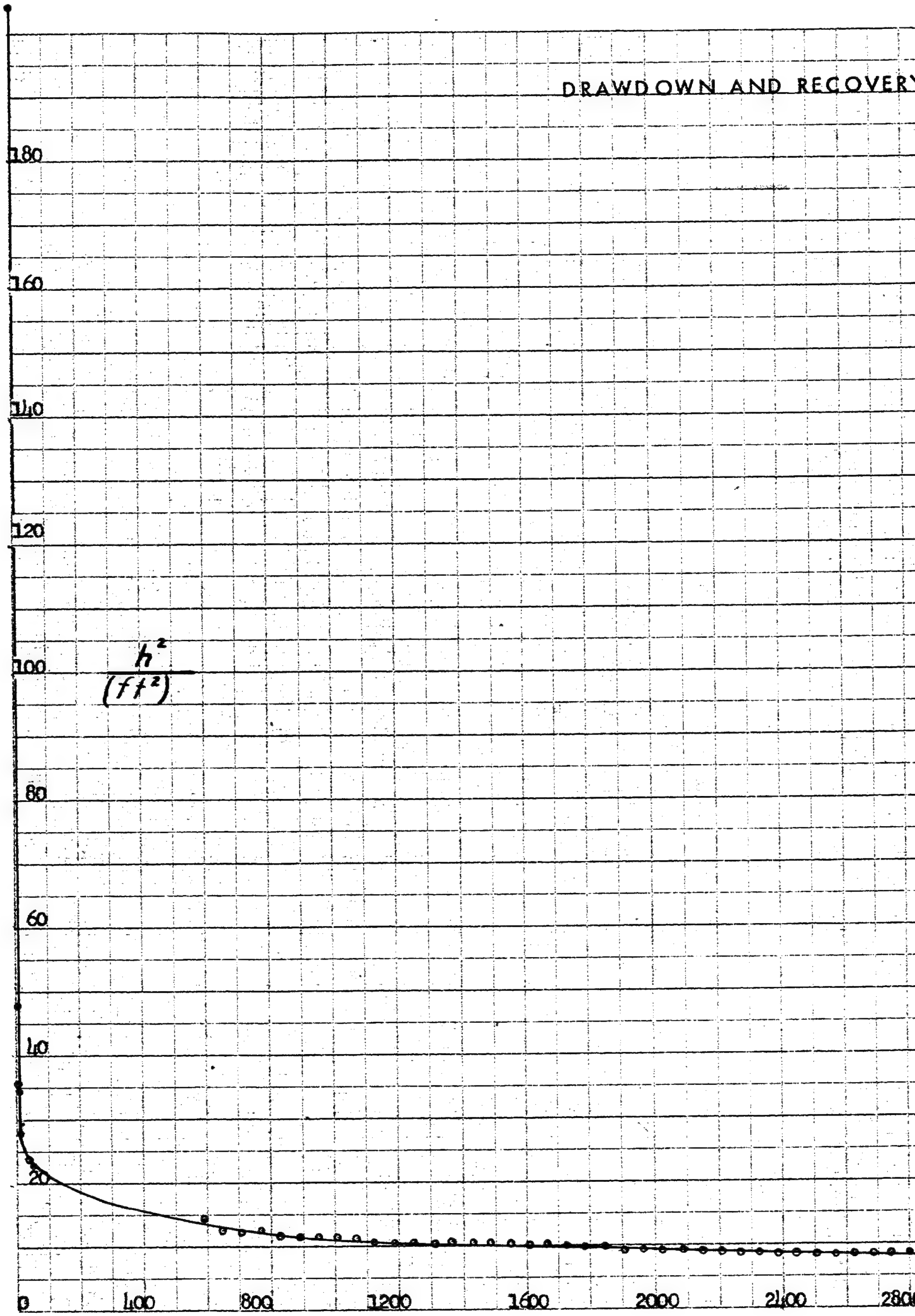


GRAPH SHOWING INTERFERENCE
P-4
P-5
P-6
$\frac{h^2}{(ft^2)}$
130
120
110
105
140
130
120
120
110
100
6000
6400
6800
7200
7600
8000
8400
8800

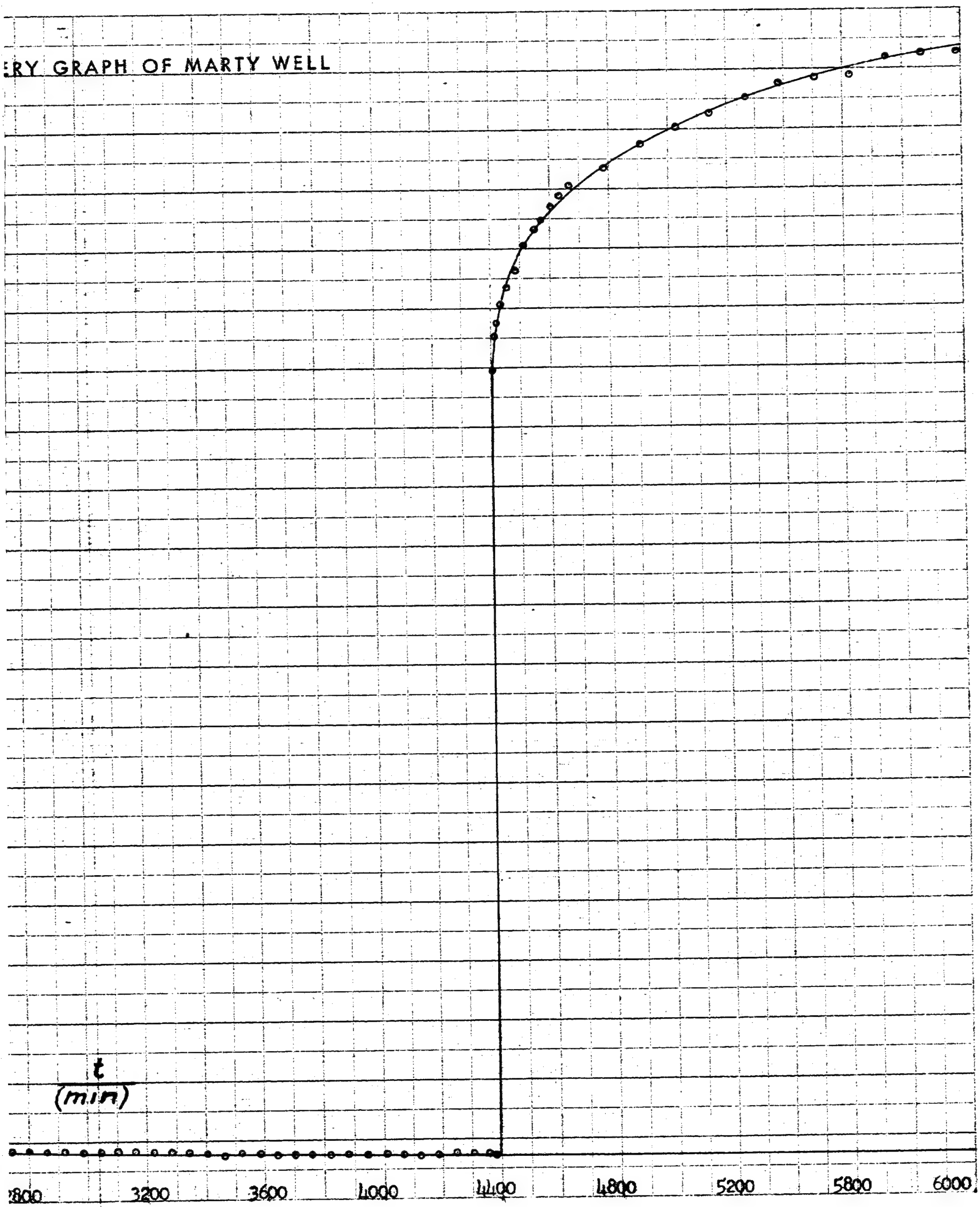
FIG. D-38b



ENCE OF ADEN WELL ON P4, P5 AND P6, Contd.
P-4
P-5
P-6
t (min)
8800
9200
9600
10,000
10,400
10,800
11,200
11,600


DRAWDOWN AND RECOVERY
180
160
140
120
100
80
60
40
20
$\frac{h^2}{(ft^2)}$
0
400
800
1200
1600
2000
2400

F'G. D-49a



KEUFFEL & ESSER CO. N. Y.

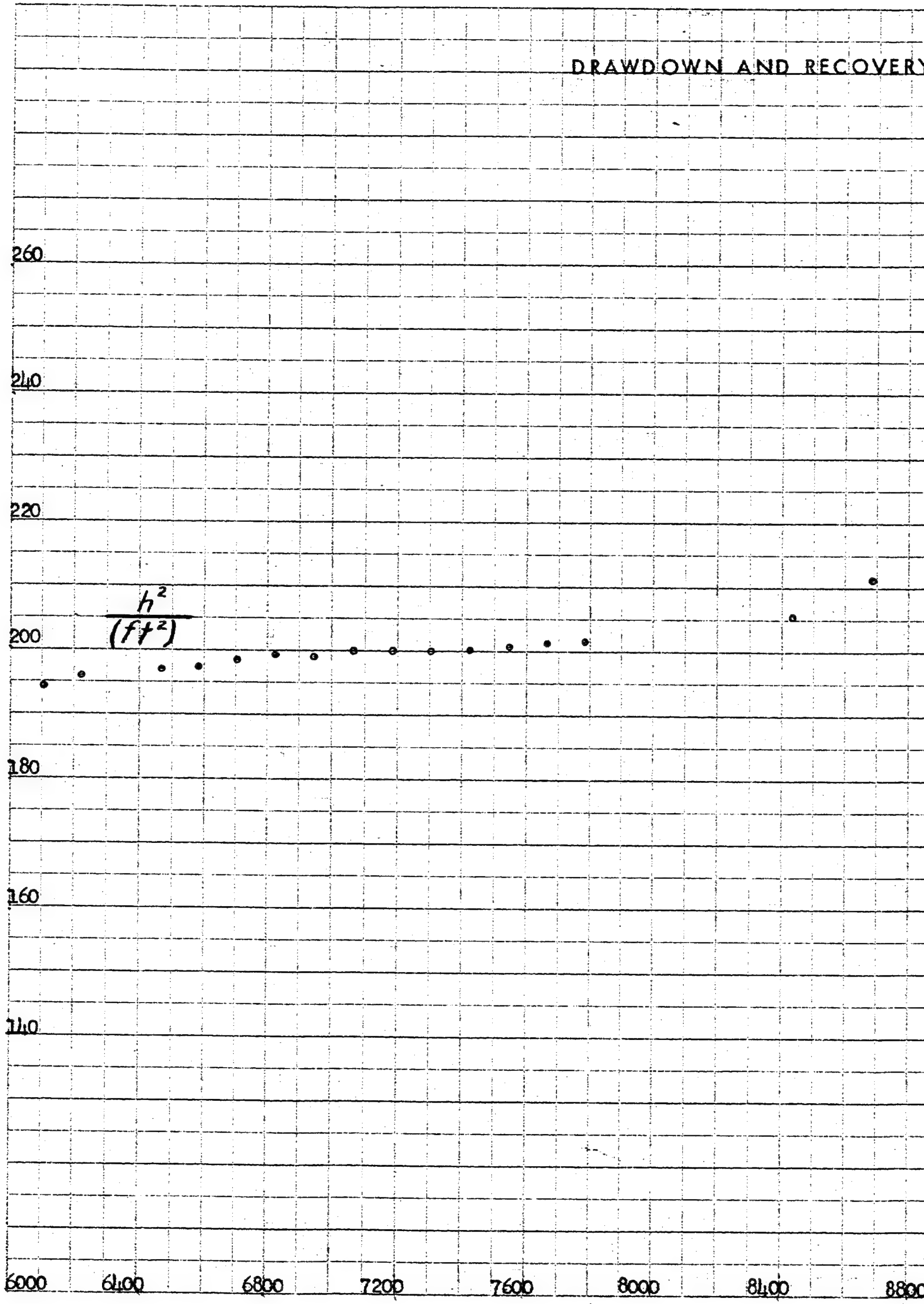
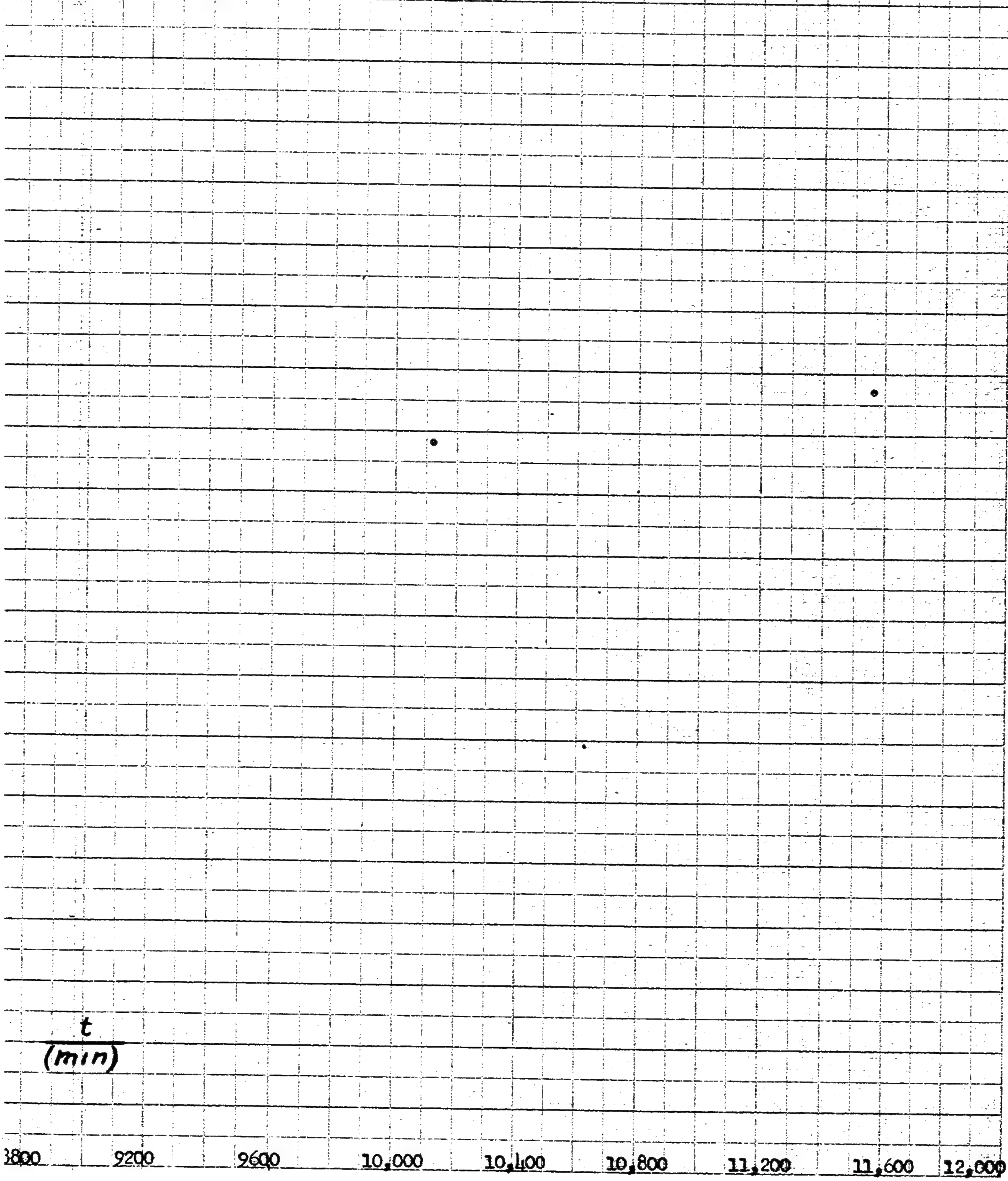

DRAWDOWN AND RECOVERY
260
240
220
200
180
160
140
h²
(ft²)
6000
6400
6800
7200
7600
8000
8400
8800

FIG. D-49b

'ERY GRAPH OF MARTY WELL, Contd.



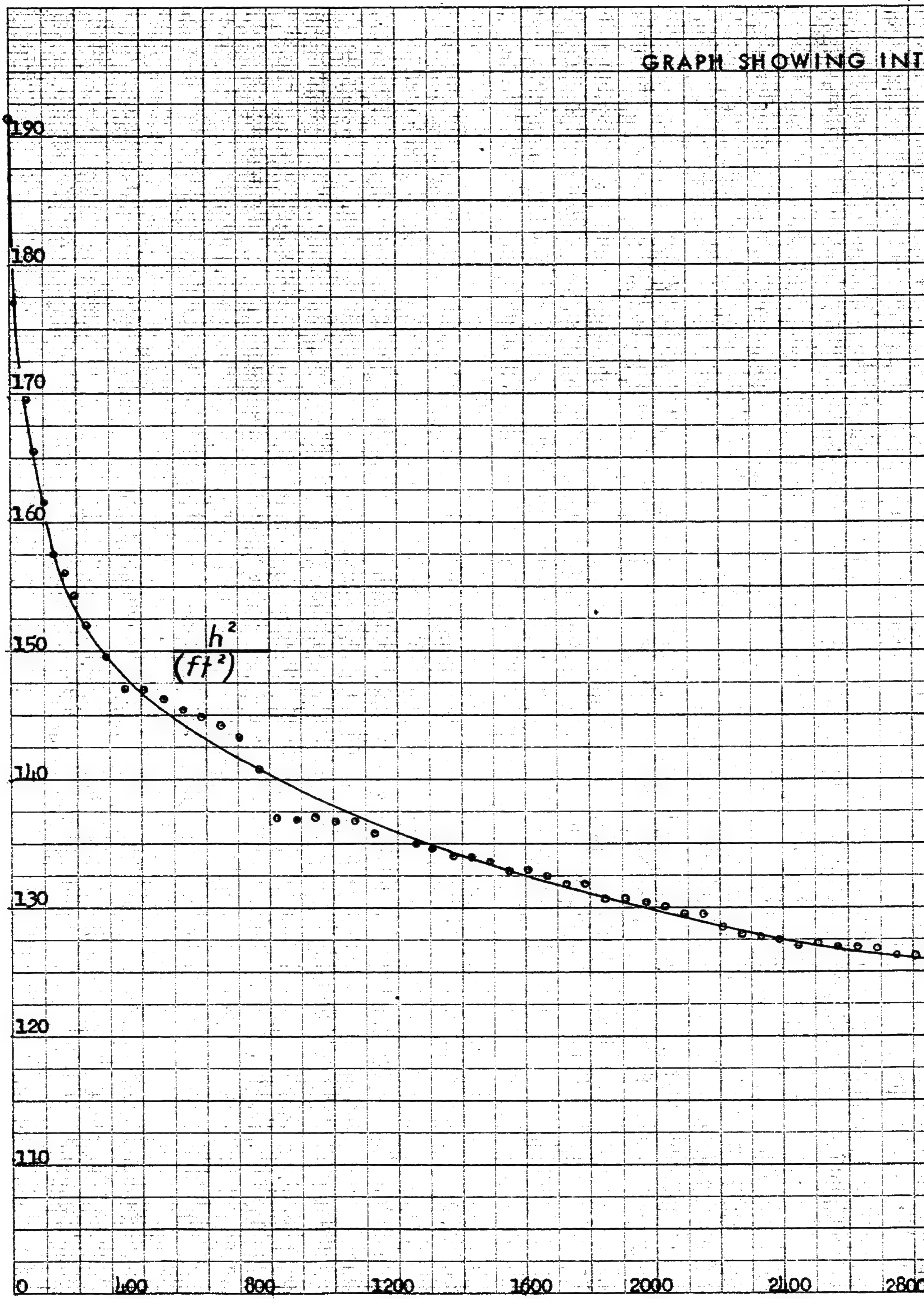
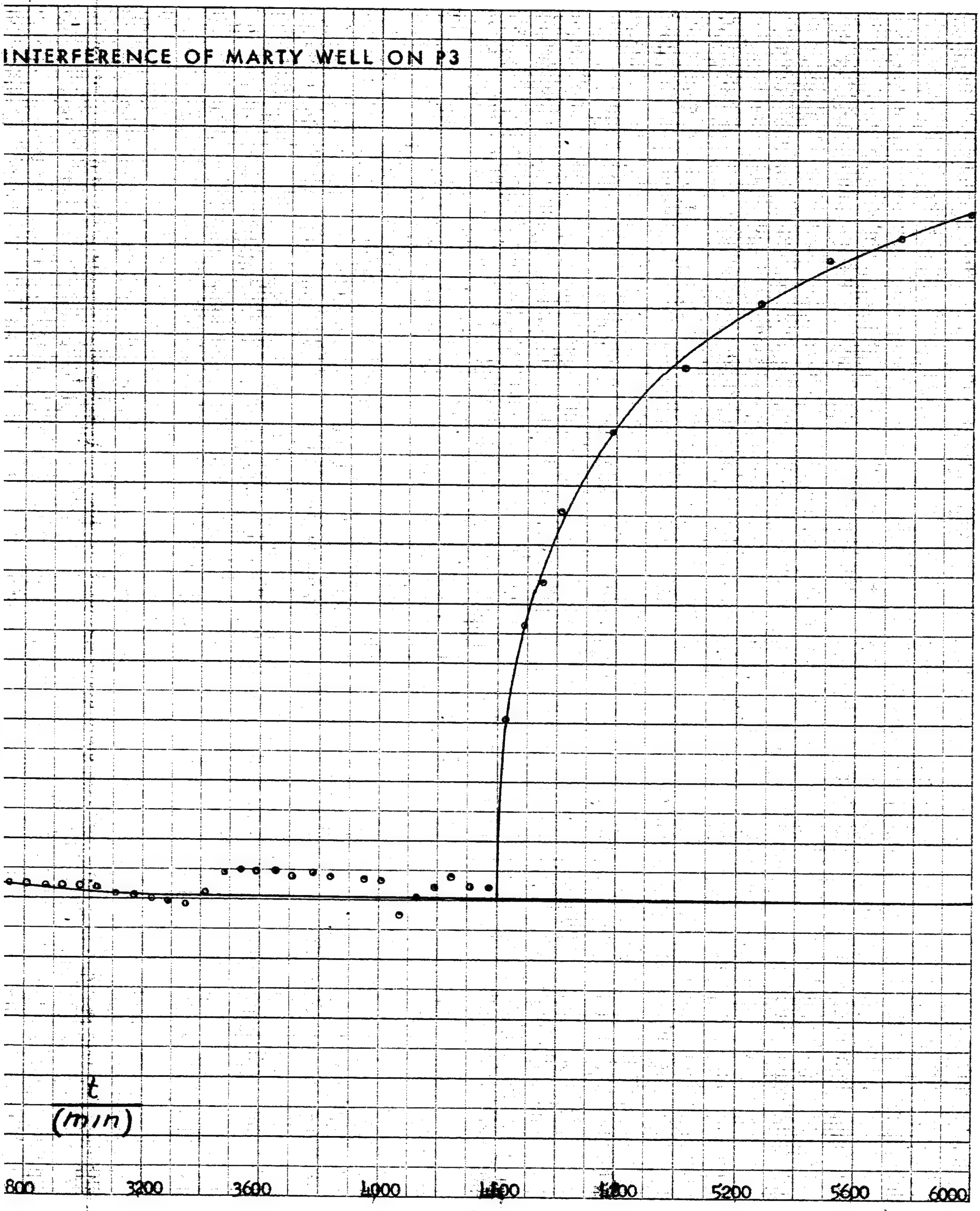

GRAPH SHOWING INT
190
180
170
160
150
140
130
120
110
$\frac{h^2}{(ft^2)}$
0
400
800
1200
1600
2000
2400
2800

FIG. D-50a


INTERFERENCE OF MARTY WELL ON P3
t
(min)
800
3200
3600
4000
4400
4800
5200
5600
6000
KEUFFEL & ESSER CO. N. Y.

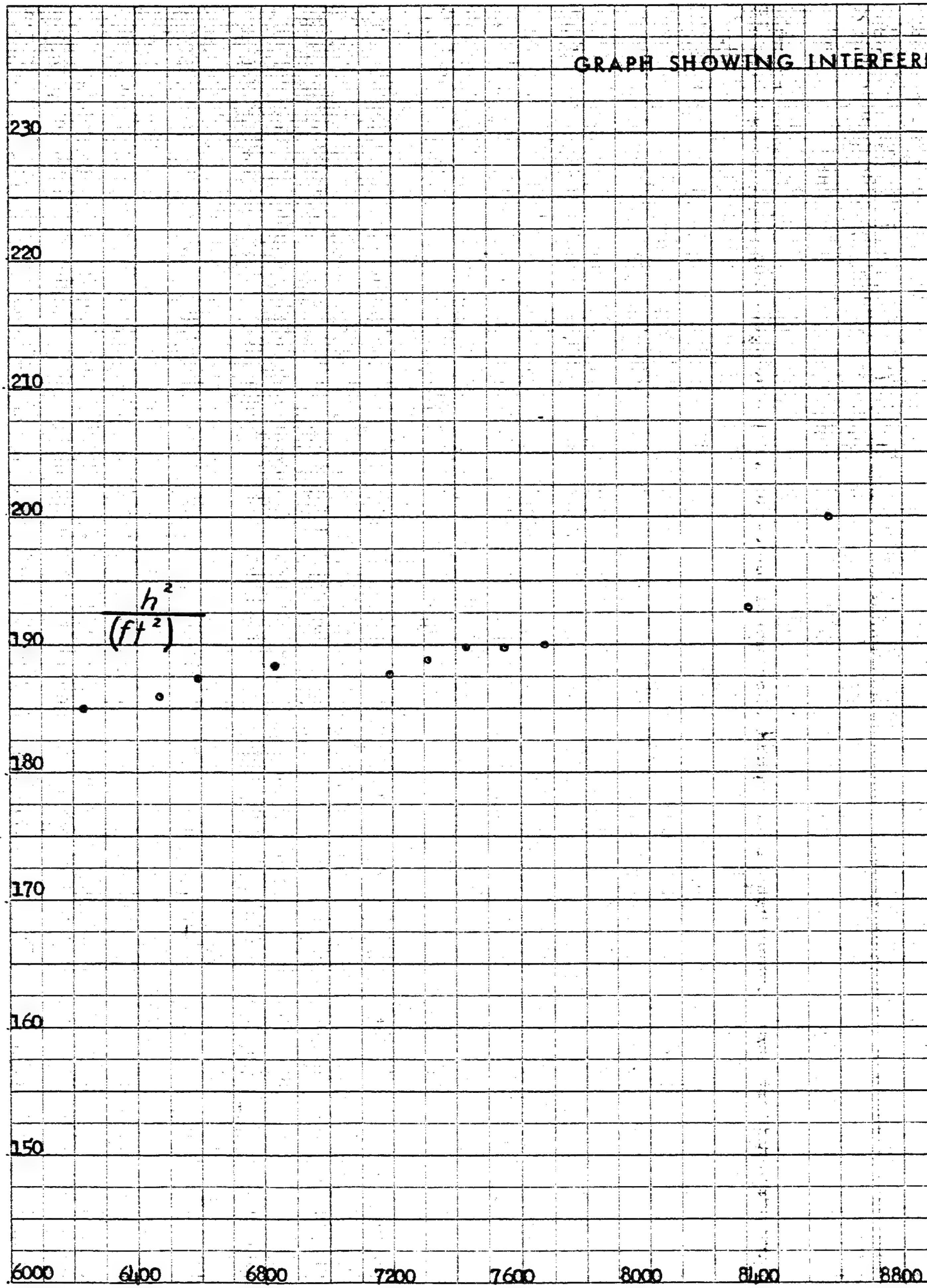
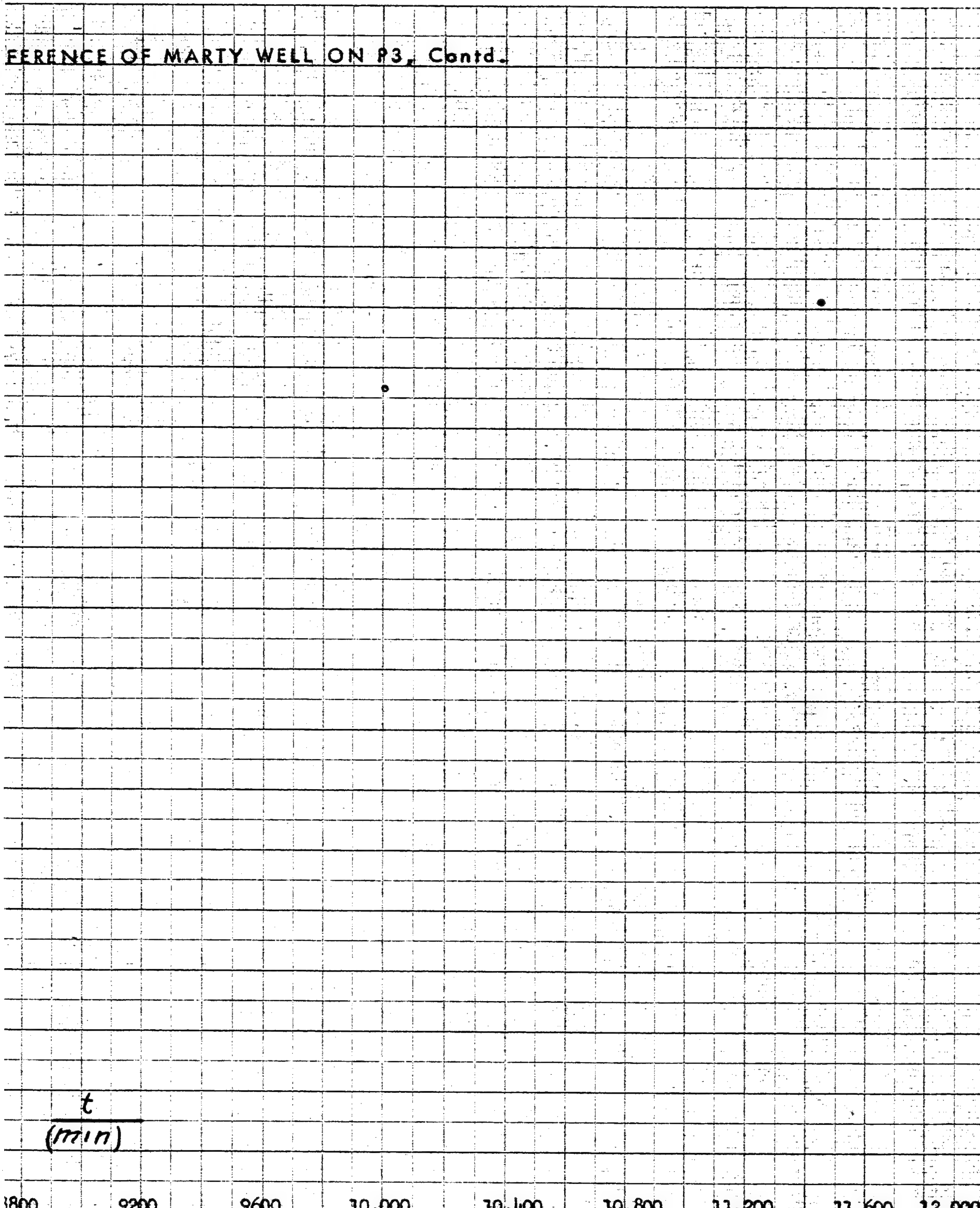
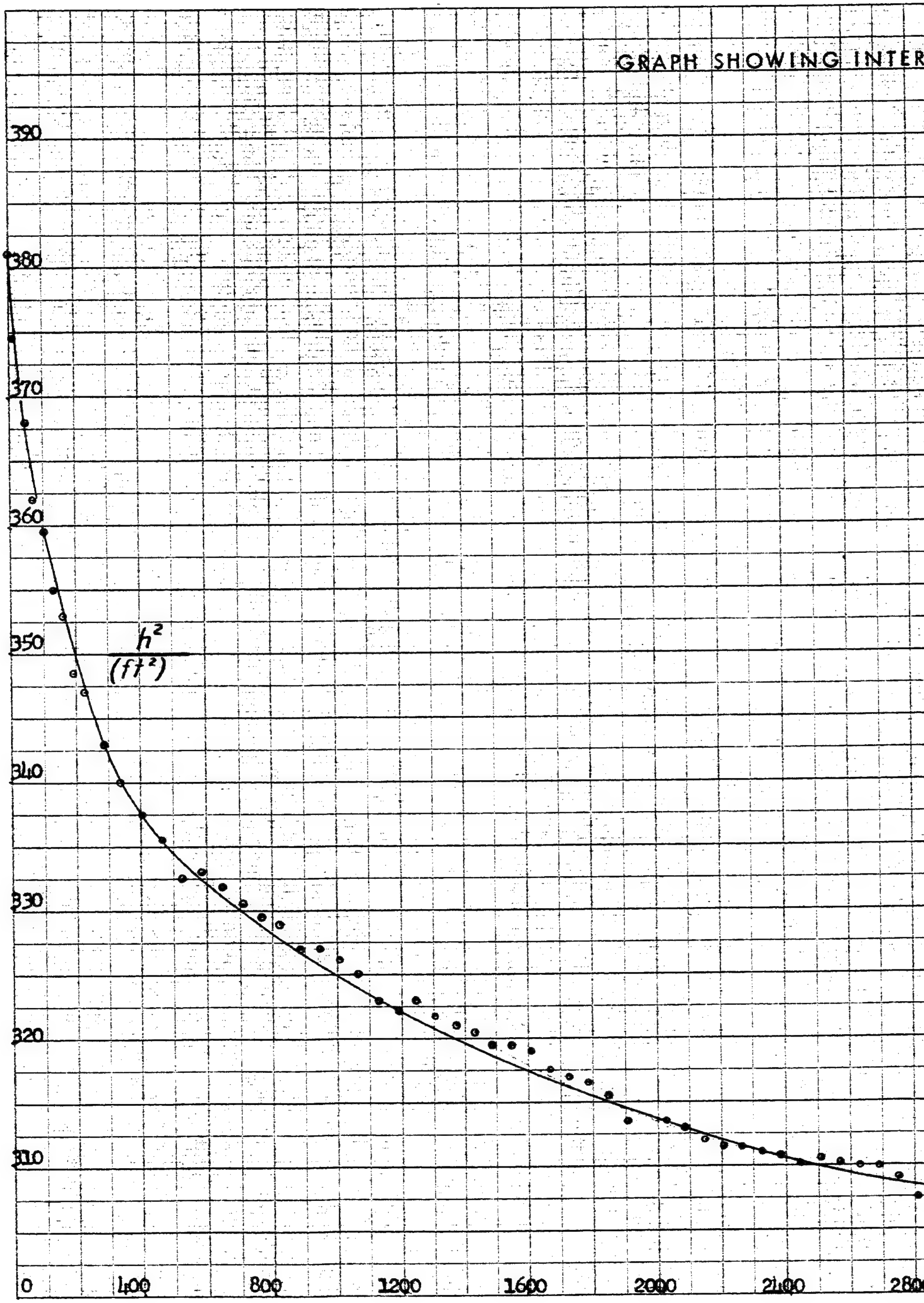
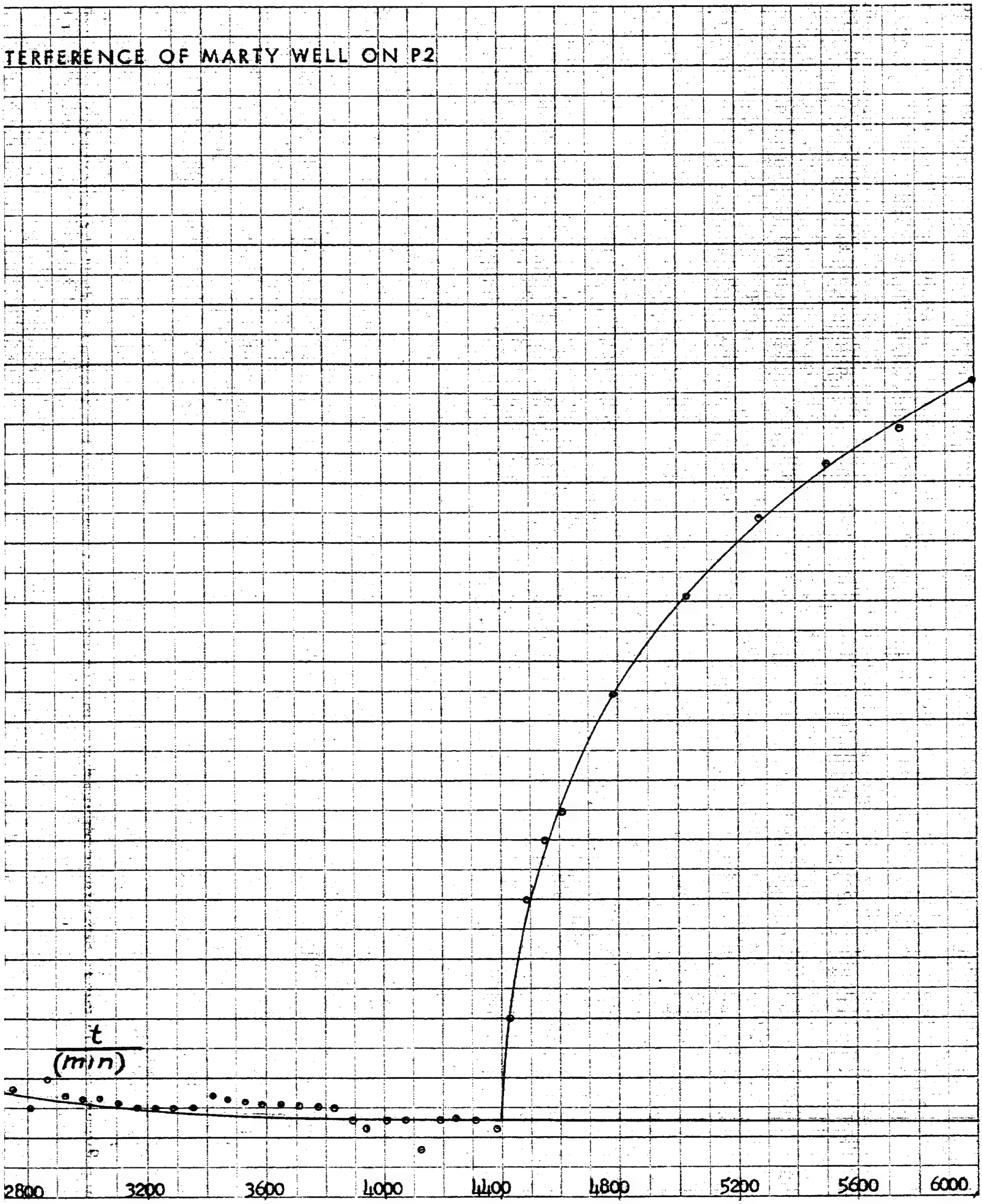

GRAPH SHOWING INTERFERE
230
220
210
200
190
180
170
160
150
$\frac{h^2}{(ft^2)}$
6000
6400
6800
7200
7600
8000
8400
8800

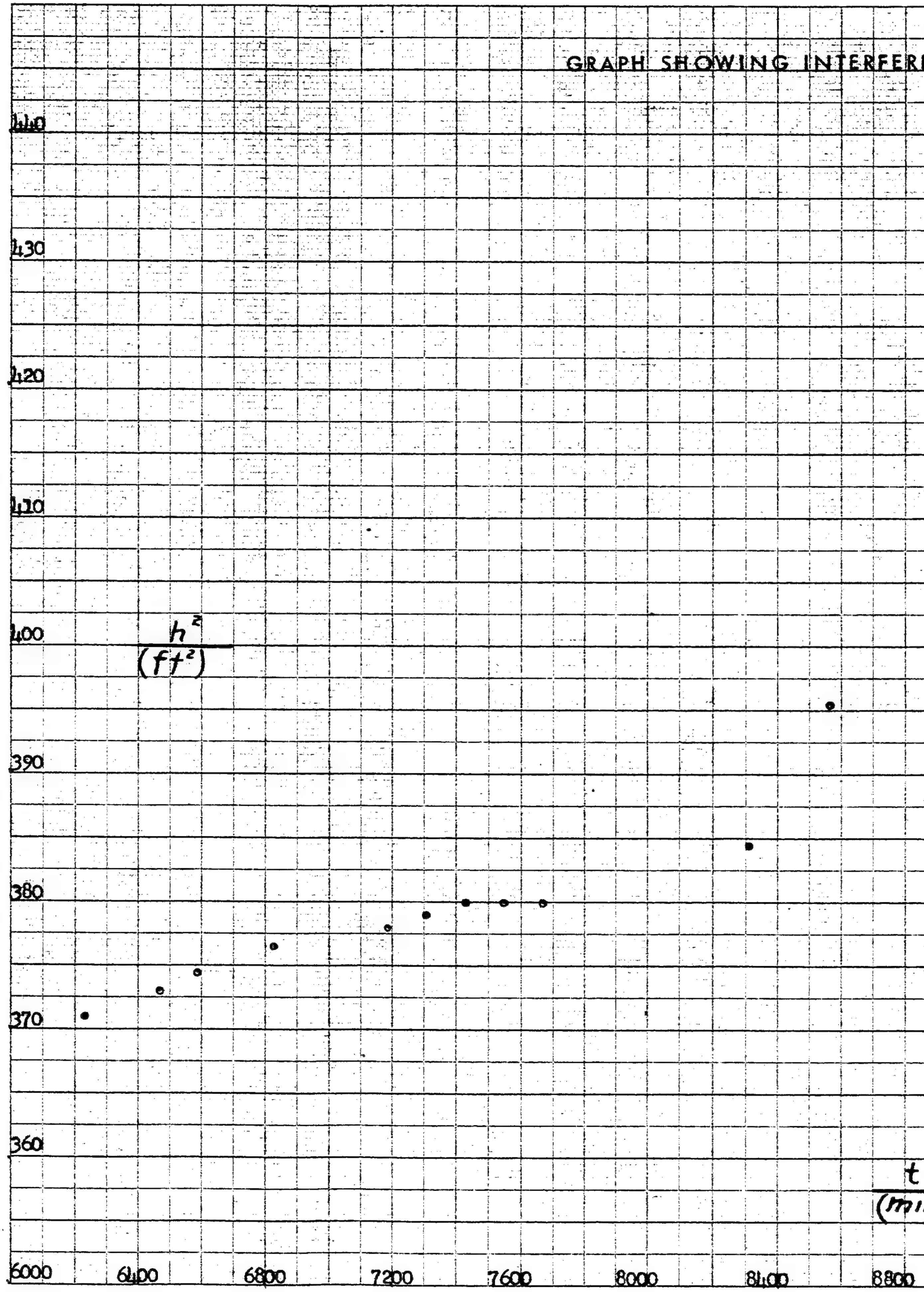
FIG. D-50b



KEUFFEL & ESSER CO. N. Y.


GRAPH SHOWING INTER
390
380
370
360
350
340
330
320
310
h²
(ft²)
0
400
800
1200
1600
2000
2400

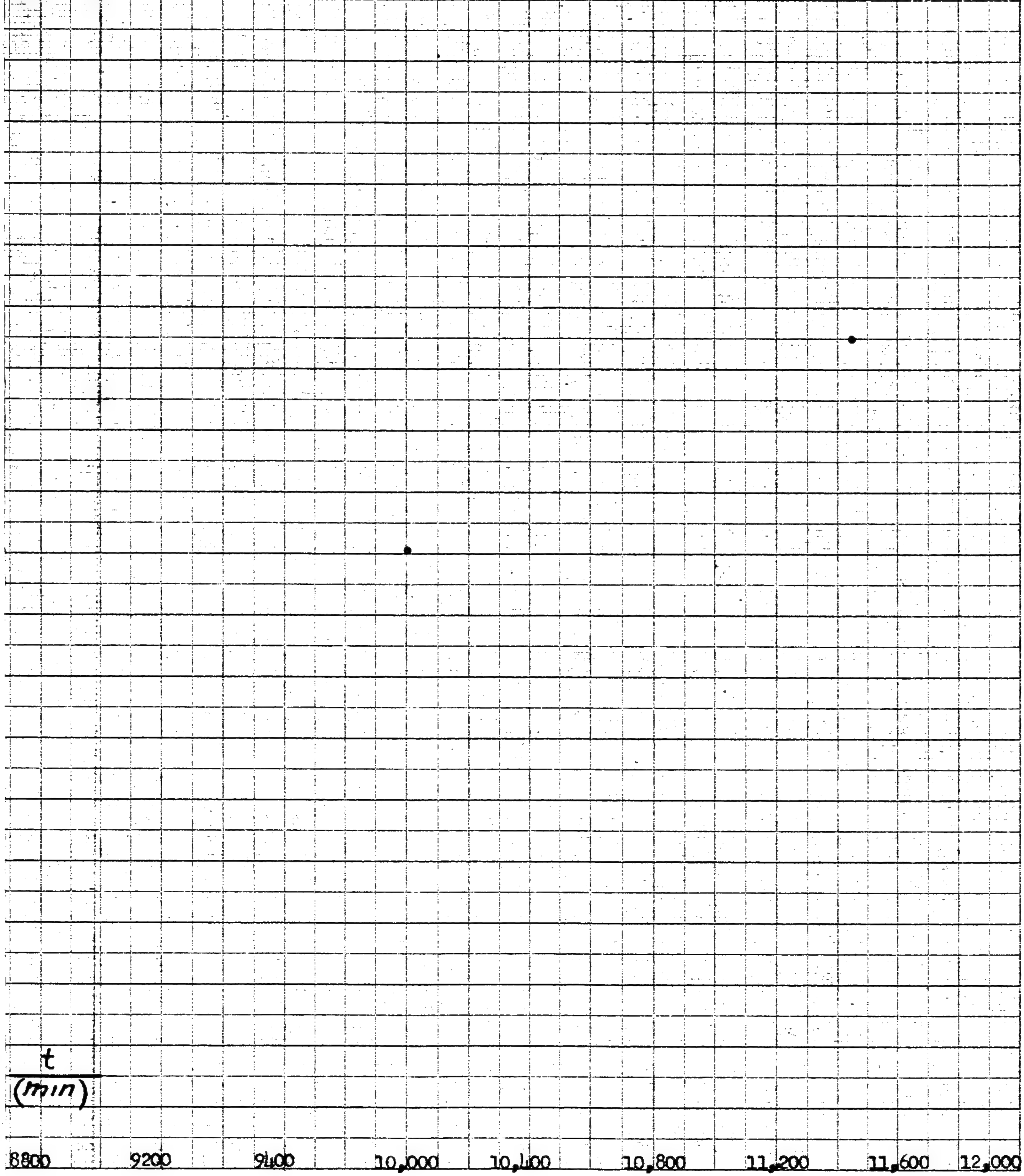

TERFERENCE OF MARTY WELL ON P2
t
(min)
2800
3200
3600
4000
4400
4800
5200
5600
6000

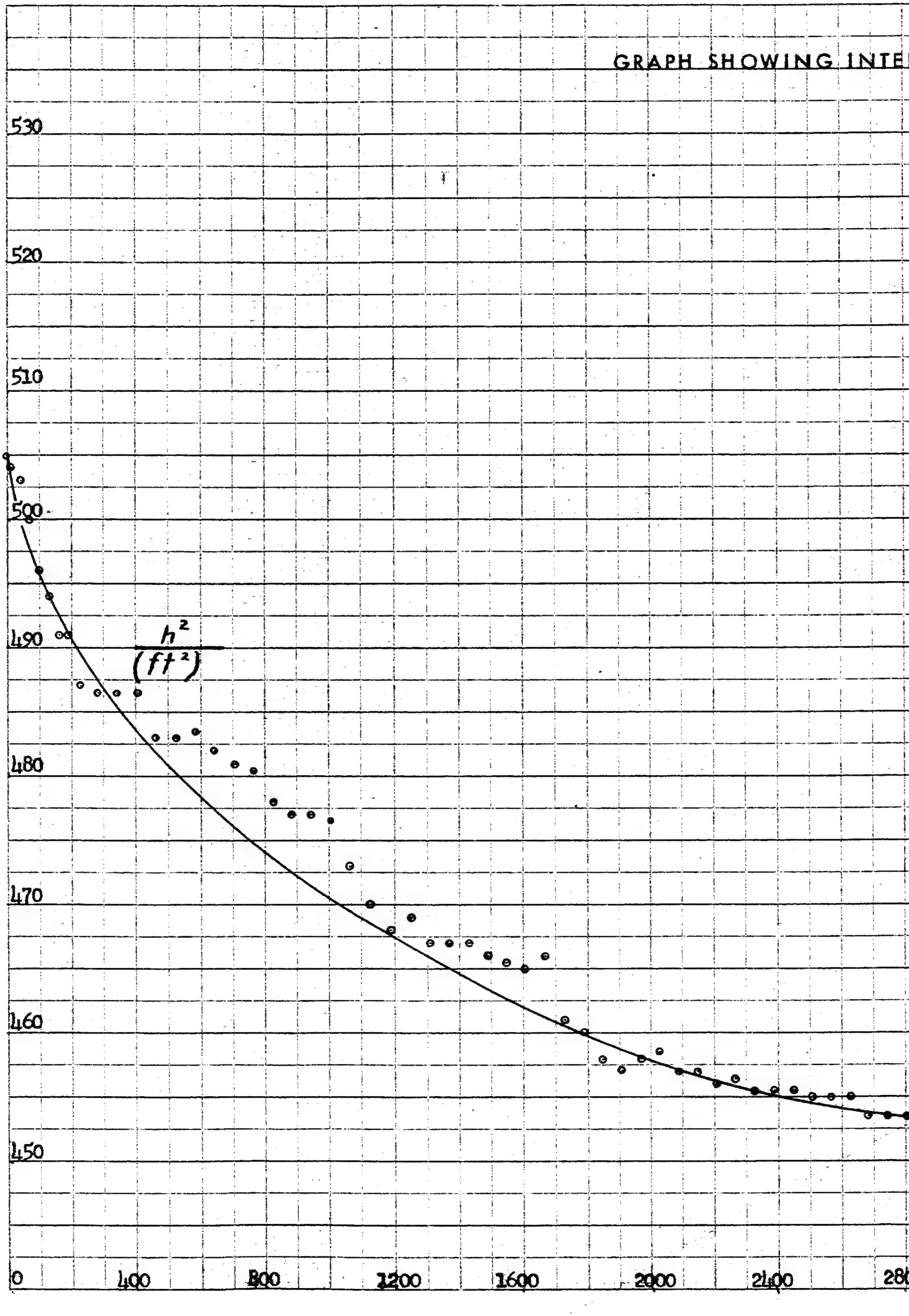


GRAPH SHOWING INTERFERE
440
430
420
410
400
390
380
370
360
h²
(ft²)
t
(mi
6000
6400
6800
7200
7600
8000
8400
8800

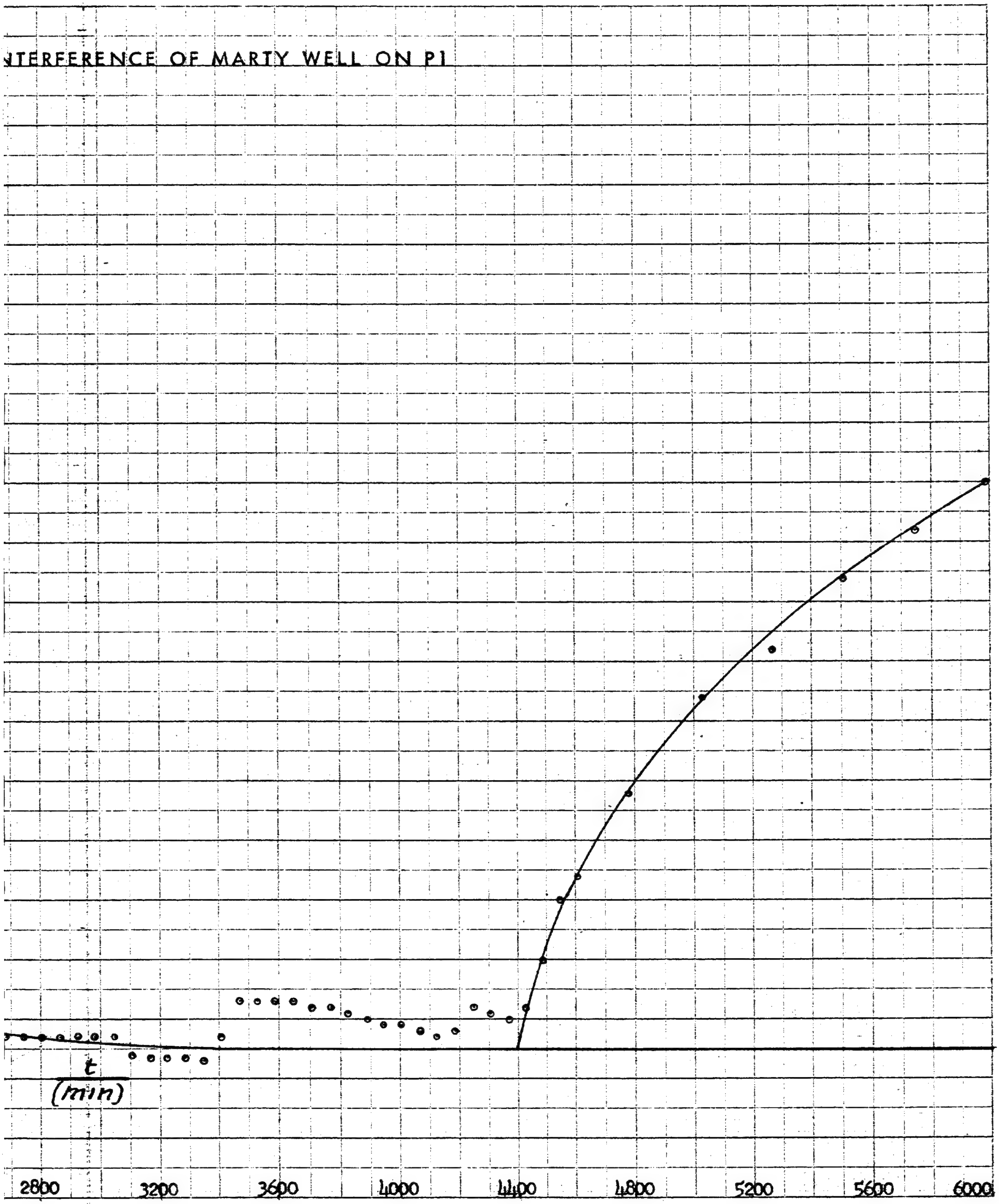
FIG. D-51b

RFERENCE OF MARTY WELL ON P2, Contd.





ENGRAVING 334-..., 10 X 10 TO THE HALF INCH


NTERFERENCE OF MARTY WELL ON P1
t
(min)
2800
3200
3600
4000
4400
4800
5200
5600
6000

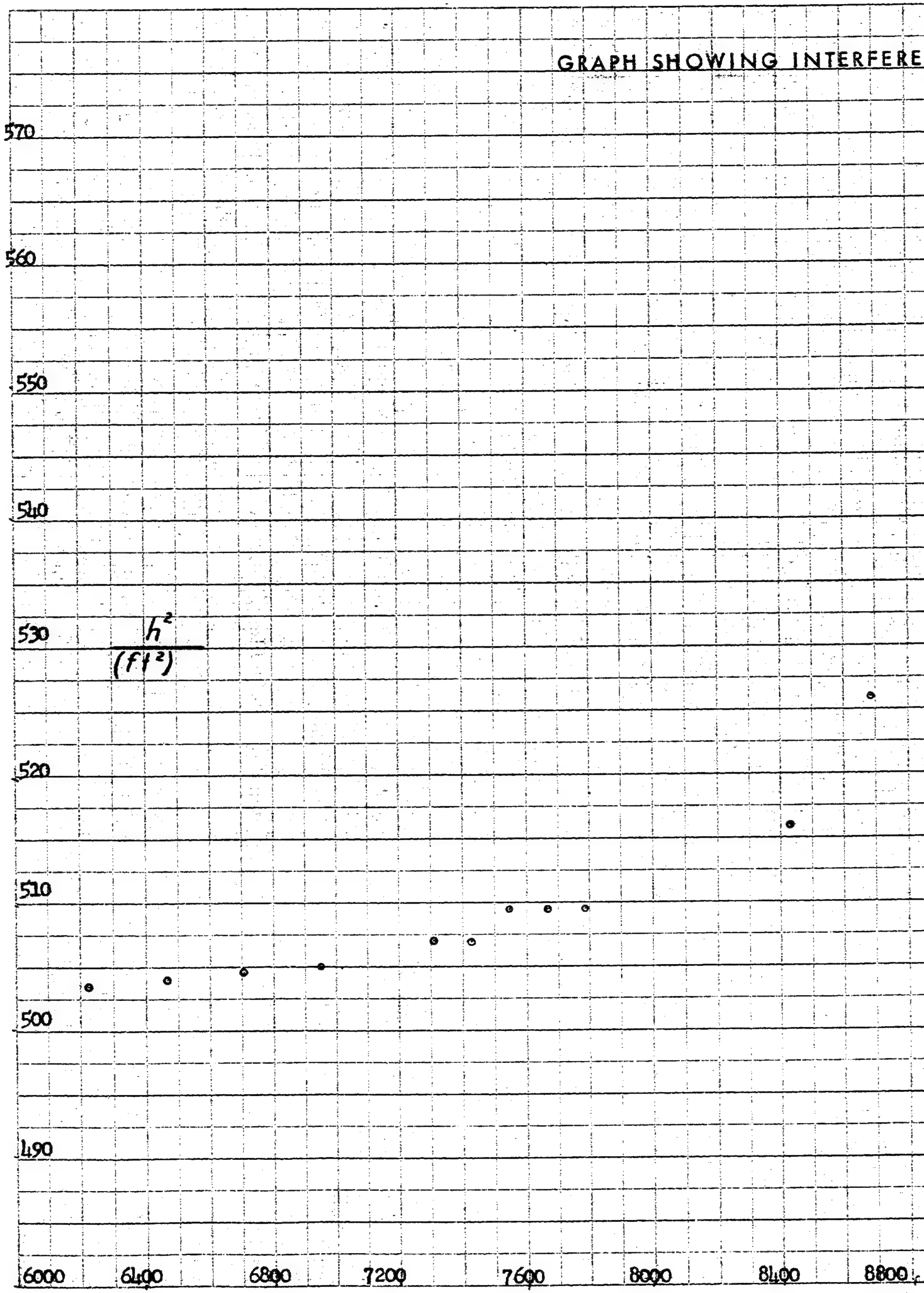
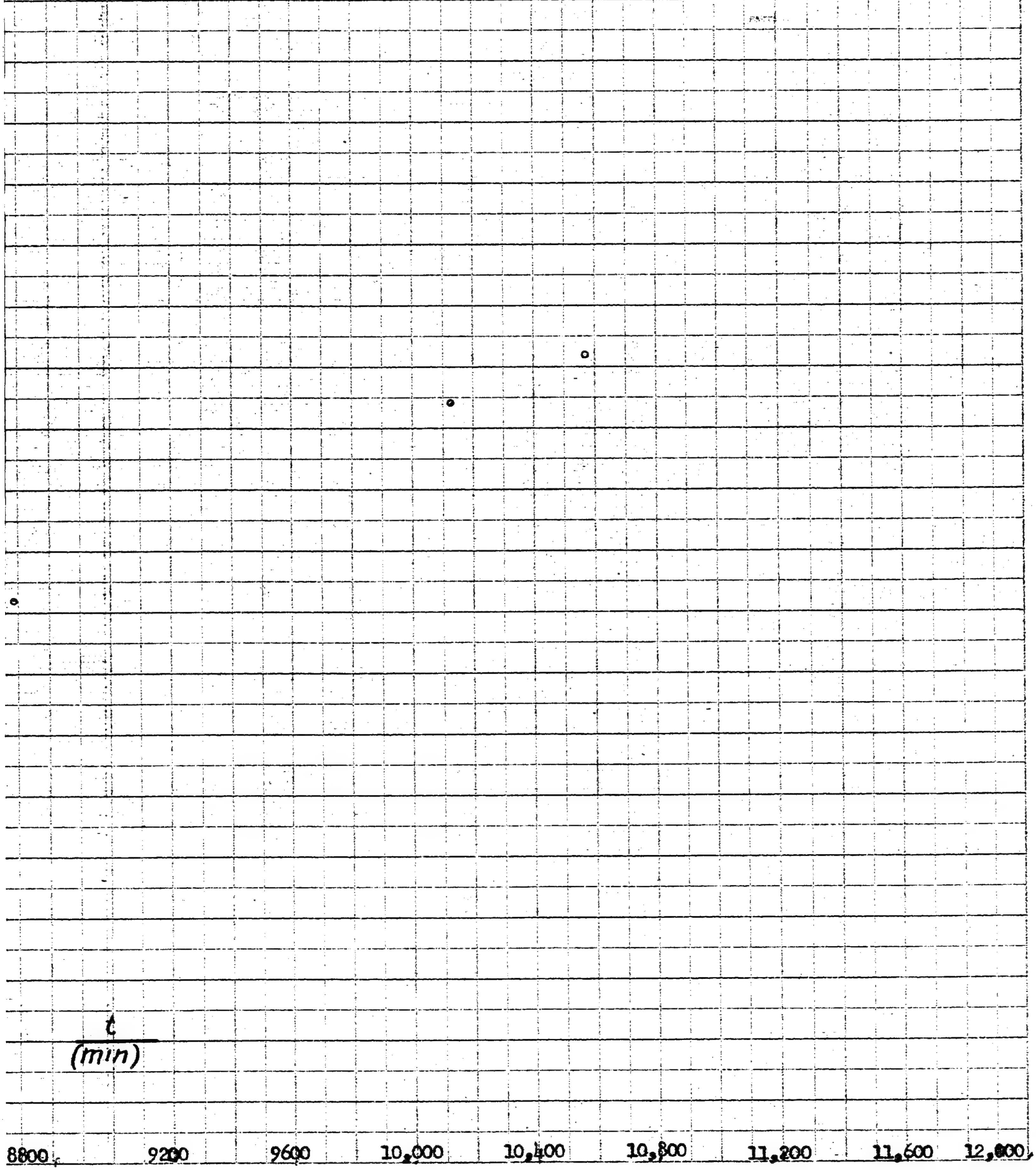
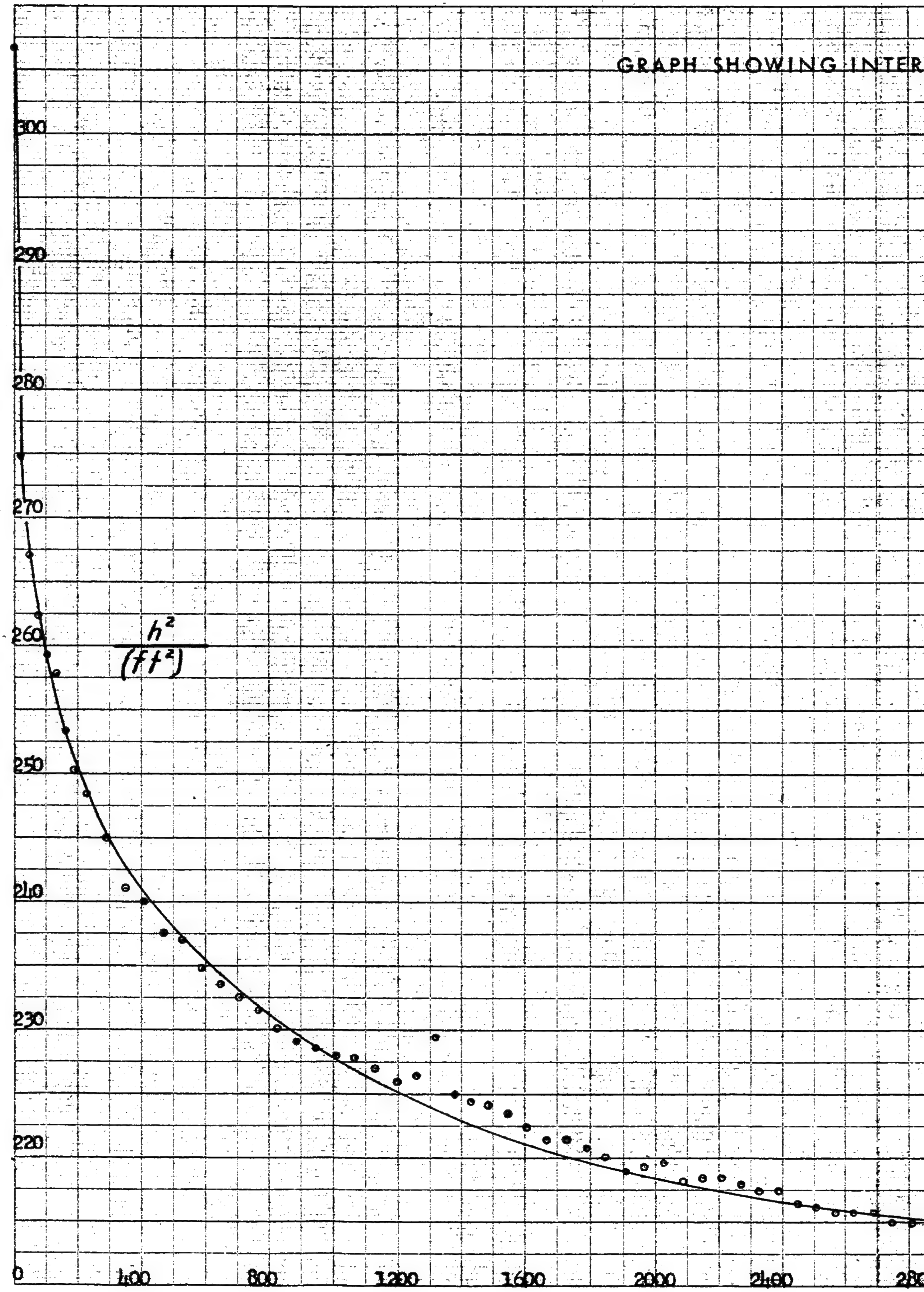
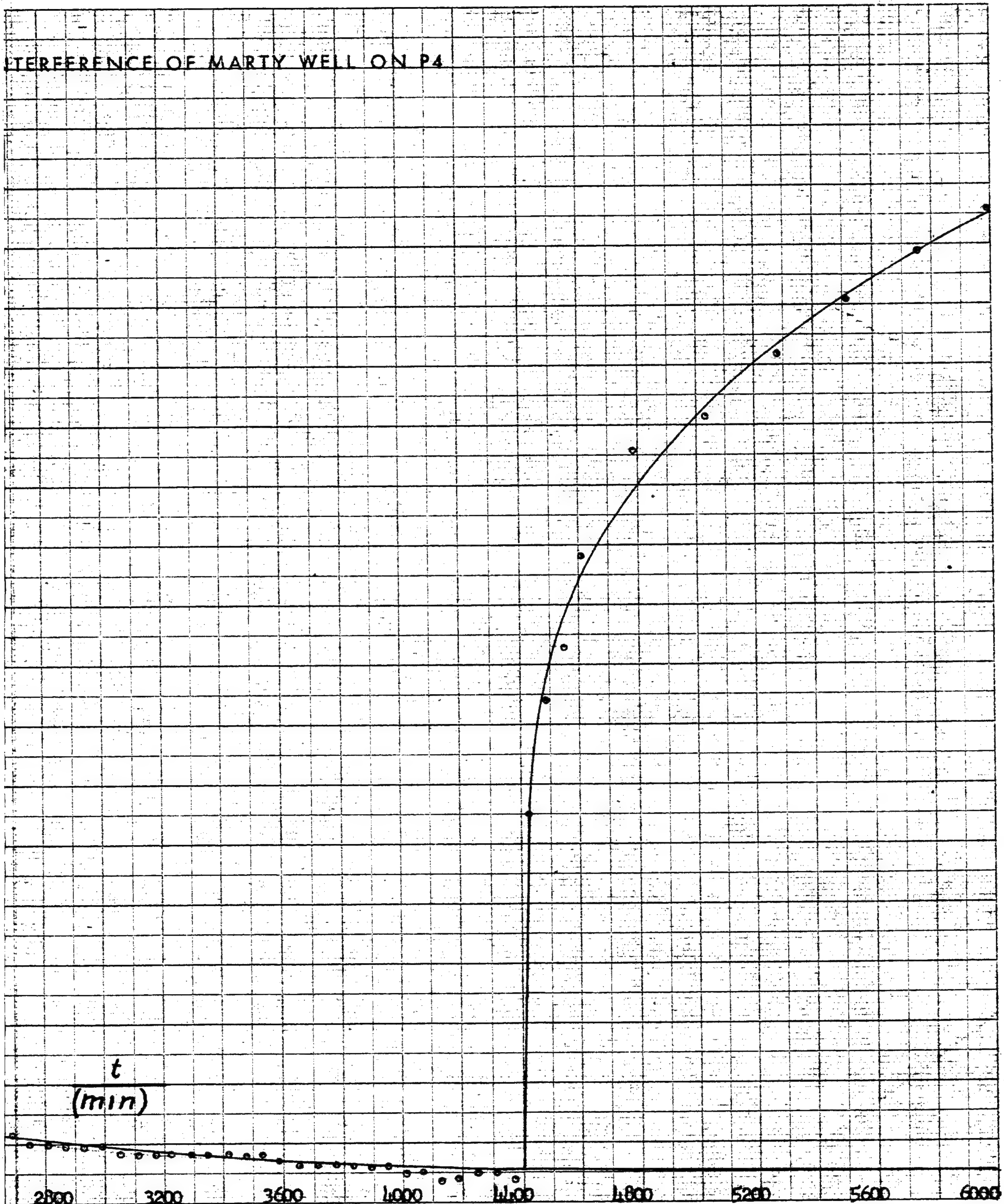

GRAPH SHOWING INTERFERE
570
560
550
540
530
520
510
500
490
$\frac{h^2}{(f+^2)}$
6000
6400
6800
7200
7600
8000
8400
8800

FIG. D-52b

RFERENCE OF MARTY WELL ON P1, Contd.




GRAPH SHOWING INTER
300
290
280
270
260
$\frac{h^2}{(ft^2)}$
250
240
230
220
0
400
800
1200
1600
2000
2400
280


ITERFERENCE OF MARTY WELL ON P4
t
(min)
2800
3200
3600
4000
4400
4800
5200
5600
6000

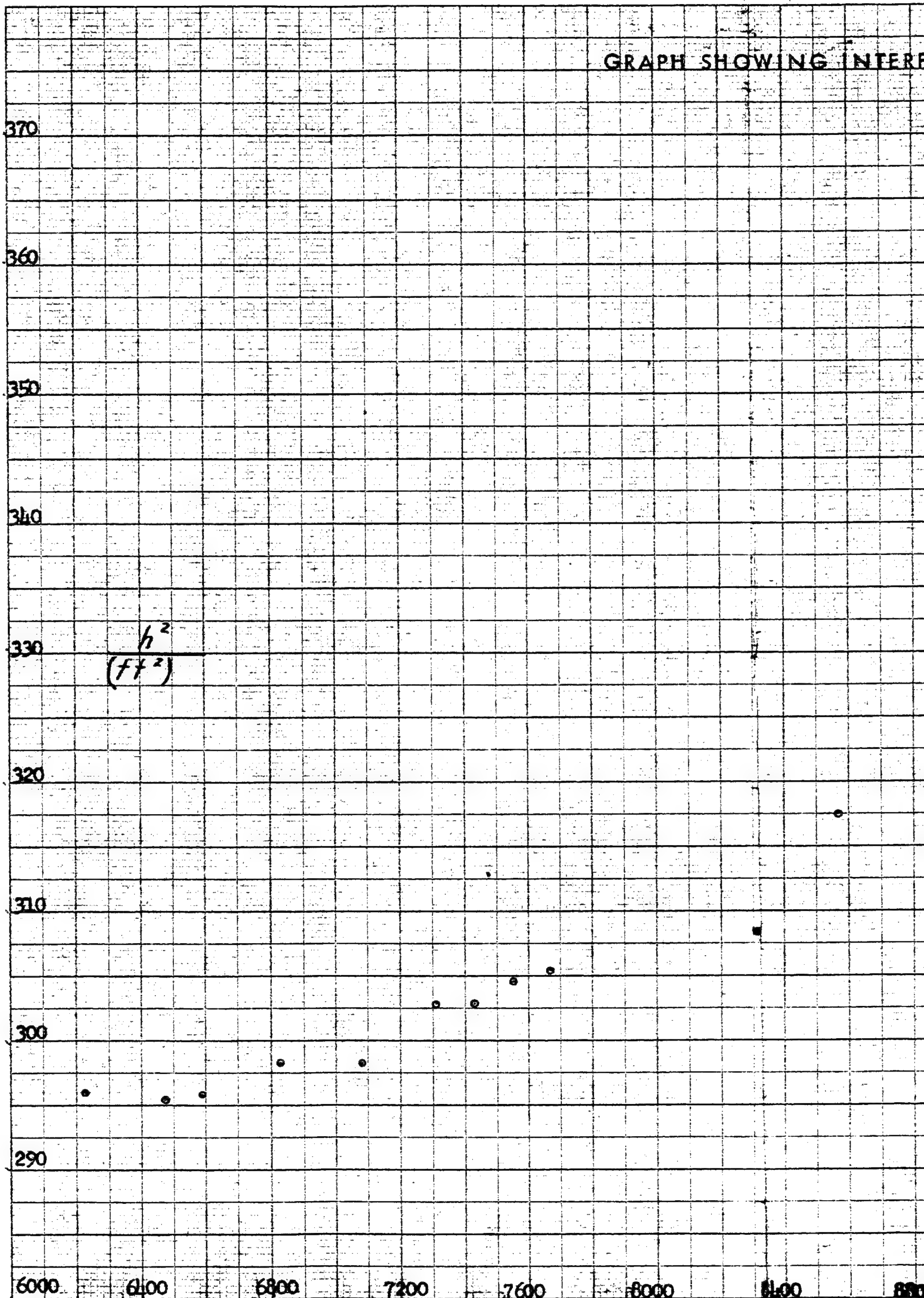
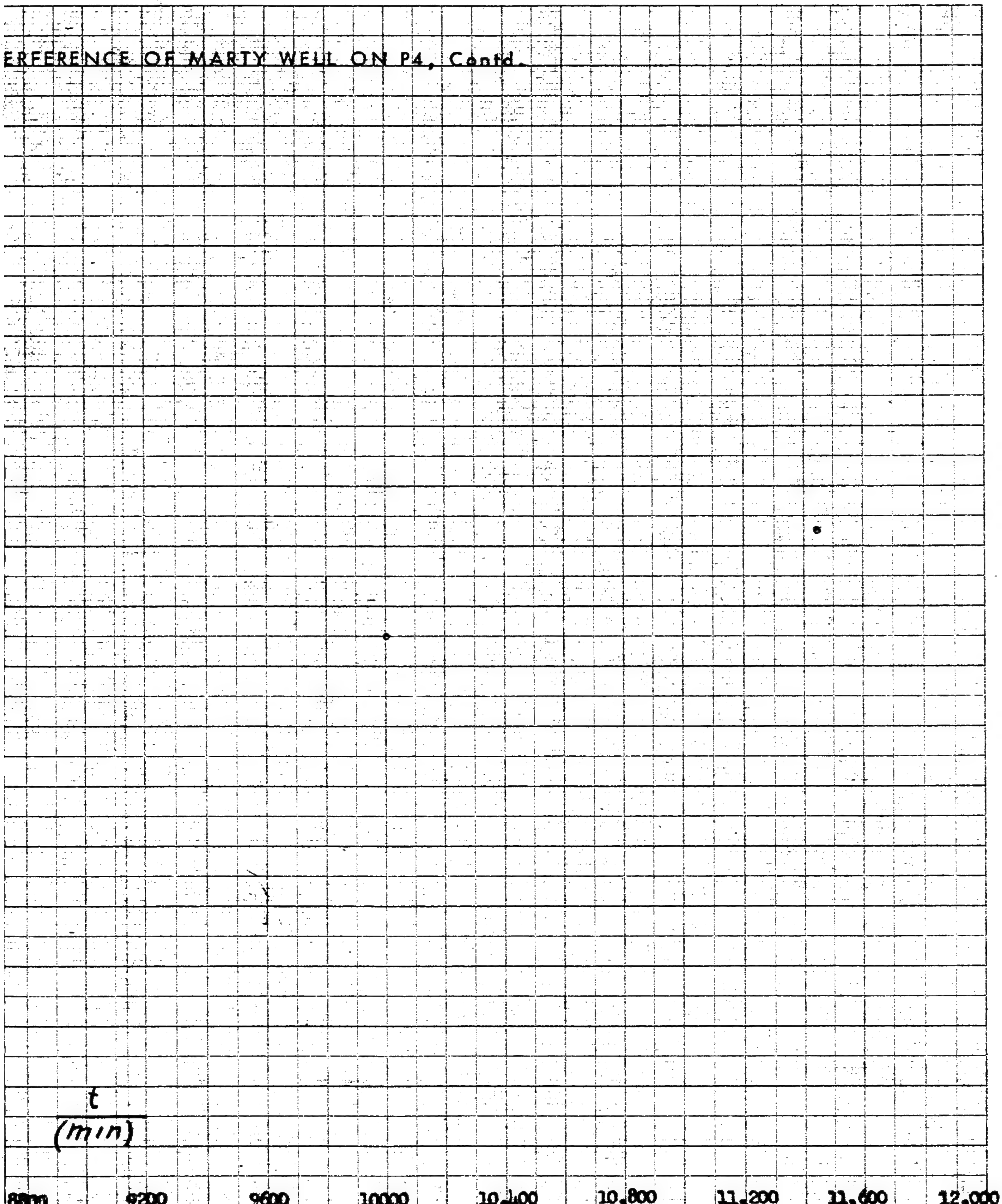
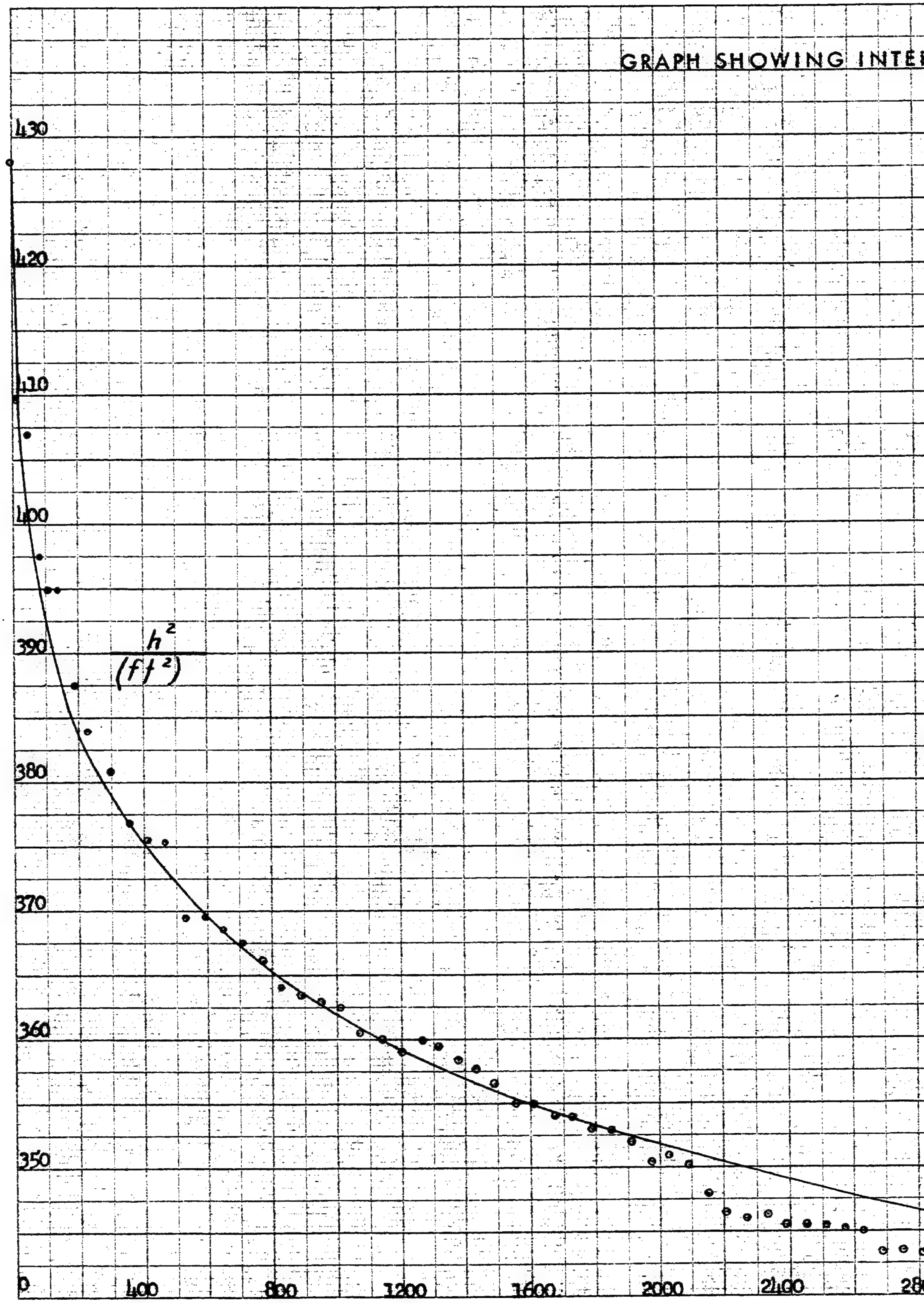

GRAPH SHOWING INTERF
370
360
350
340
330
320
310
300
290
6000
6400
6800
7200
7600
8000
8400
$\frac{h^2}{(ft^2)}$

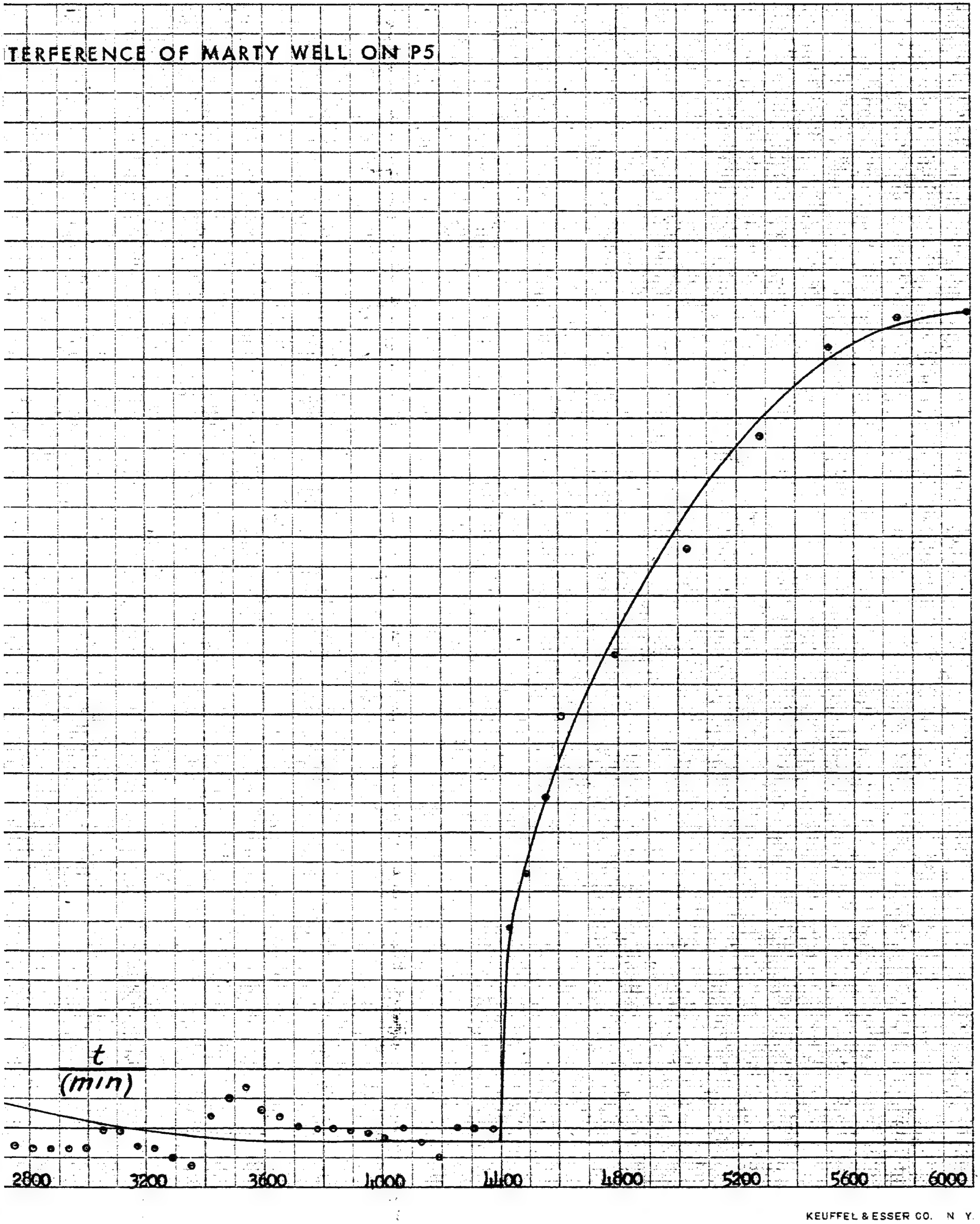
FIG. D-53b


ERFERENCE OF MARTY WELL ON P4, Contd.
t
(min)
8800
9200
9600
10000
10,400
10,800
11,200
11,600
12,000

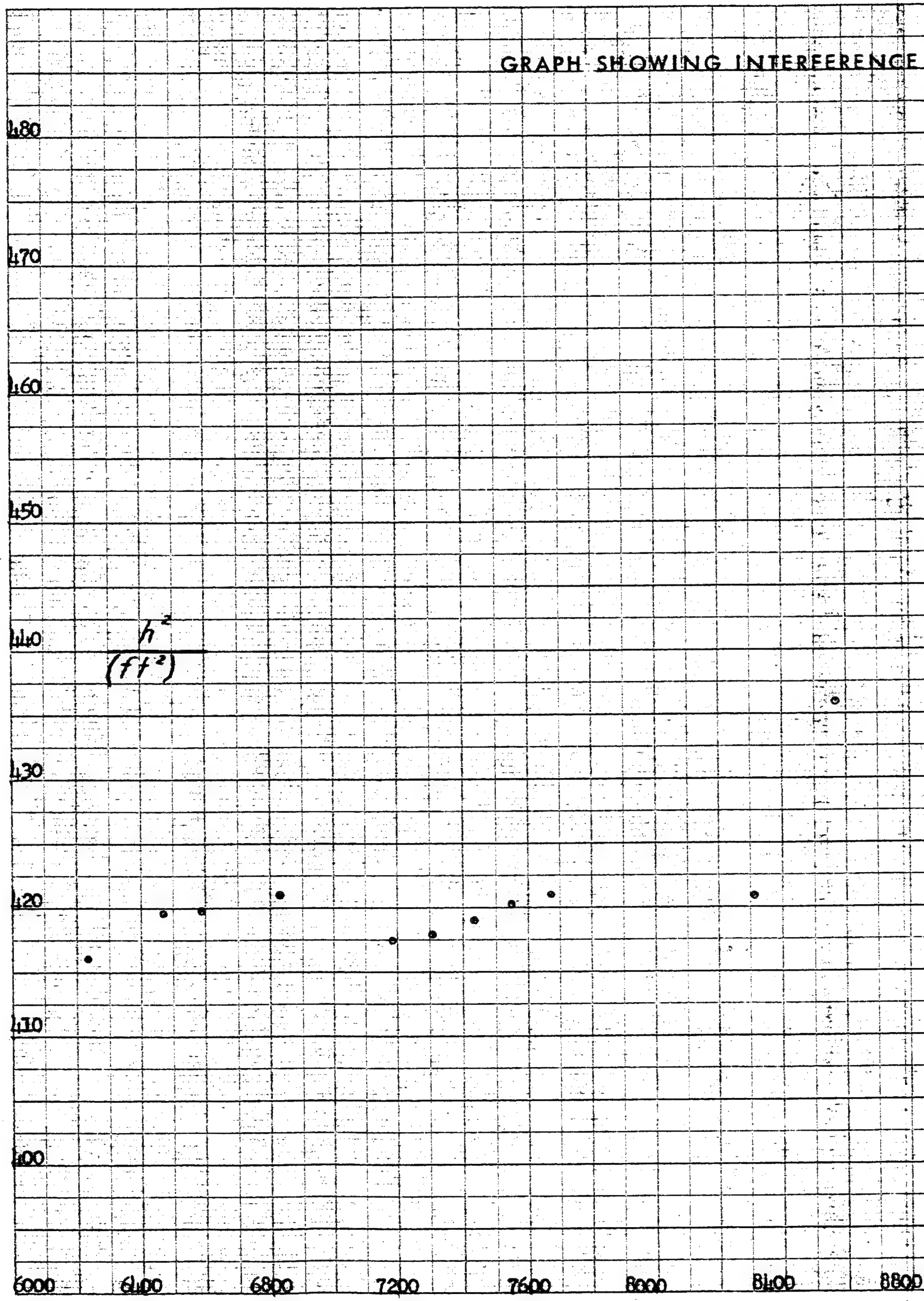


GRAPH SHOWING INTE
430
420
410
400
390
380
370
360
350
0
400
800
1200
1600
2000
2400
$\frac{h^2}{(f+^2)}$

FIG. D-54a

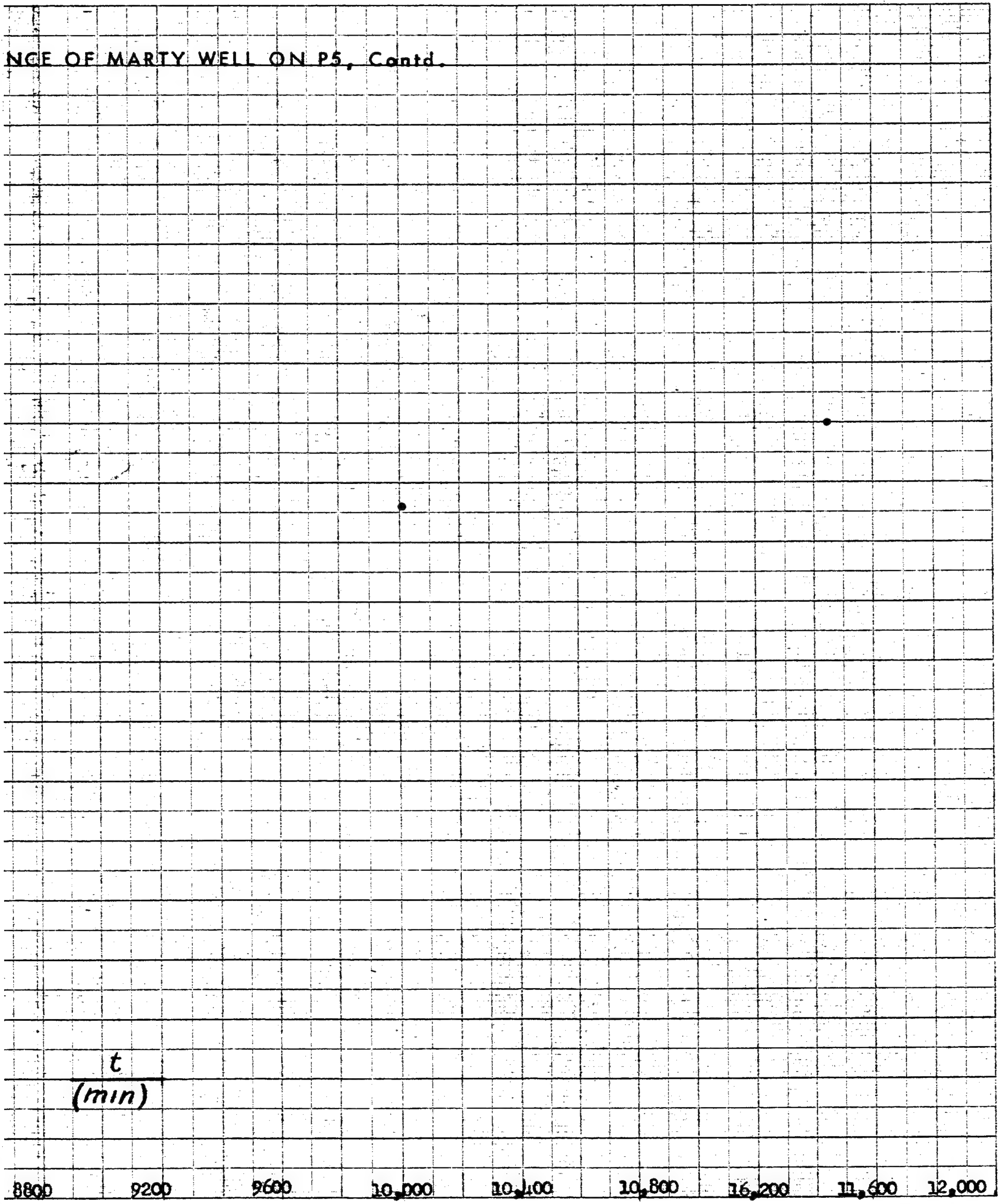


TERFERENCE OF MARTY WELL ON P5
t
(min)
2800
3200
3600
4000
4400
4800
5200
5600
6000
KEUFFEL & ESSER CO. N. Y.



GRAPH SHOWING INTERFERENCE
$\frac{h^2}{(ft^2)}$
480
470
460
450
440
430
420
410
400
6000
6400
6800
7200
7600
8000
8400
8800

FIG. D-54b



KEUFFEL & ESSER CO. N. Y.

GRAPH SHOWING INTE

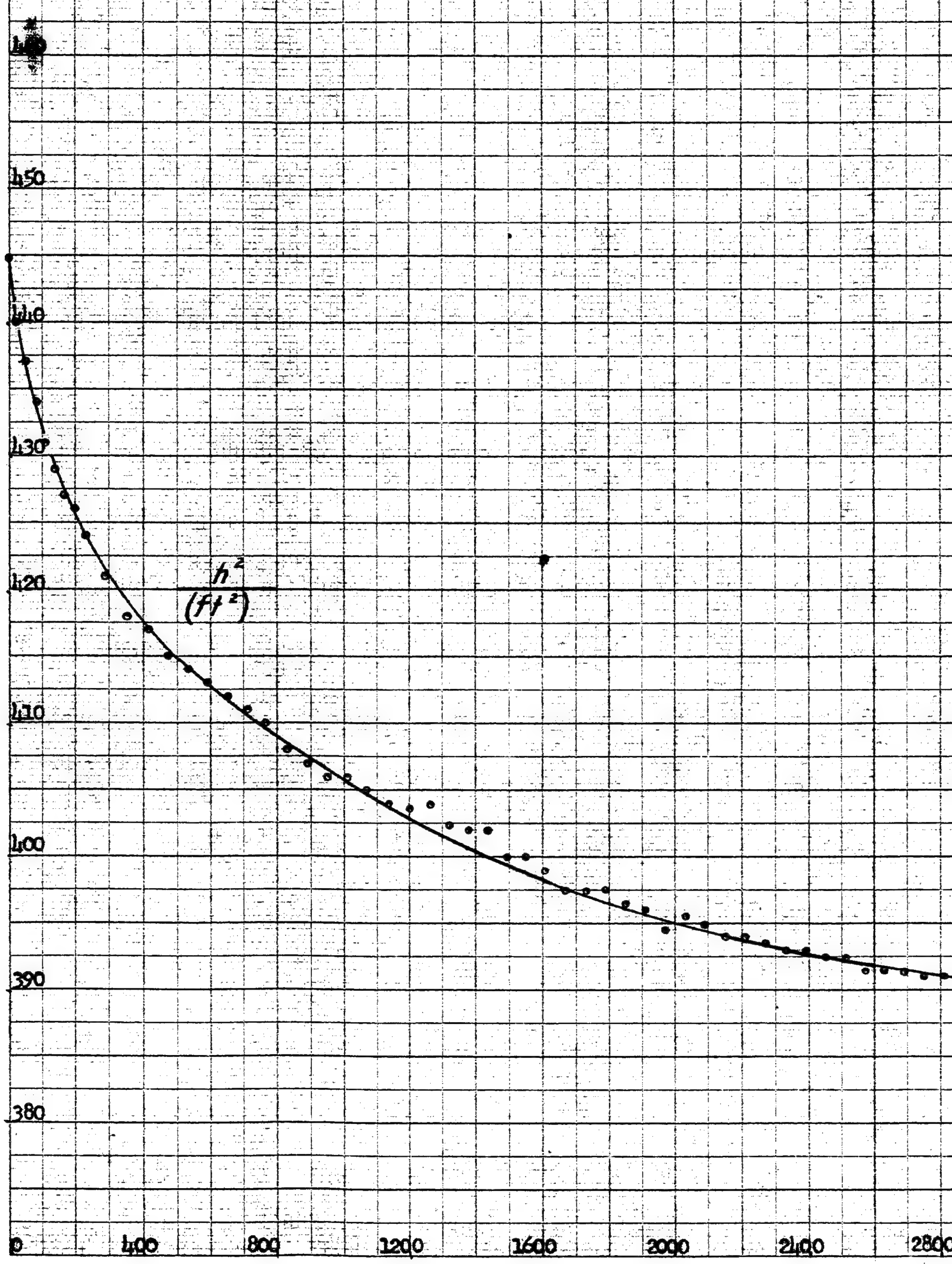
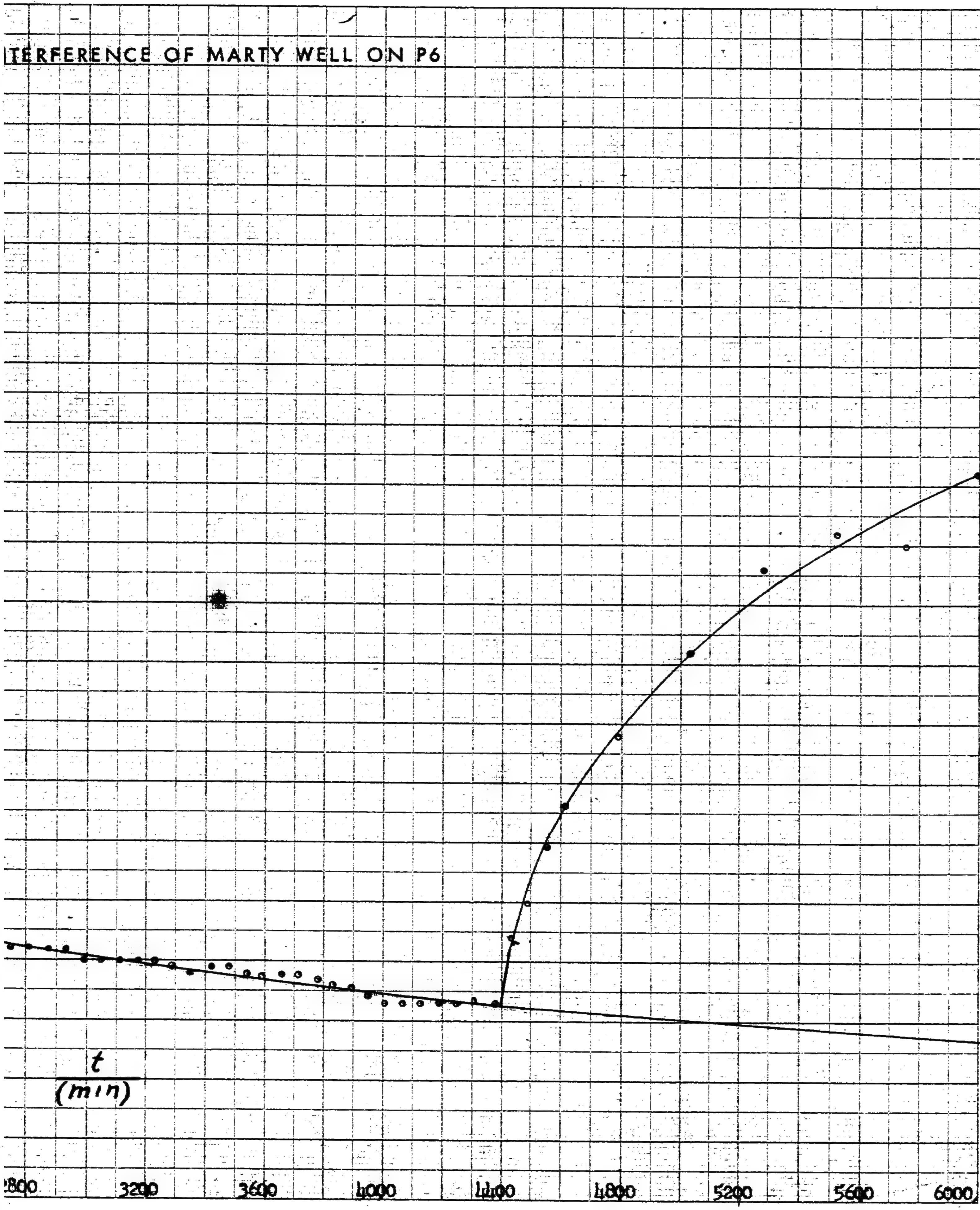


FIG. D-55a


ITERFERENCE OF MARTY WELL ON P6
t
(min)
2800
3200
3600
4000
4400
4800
5200
5600
6000

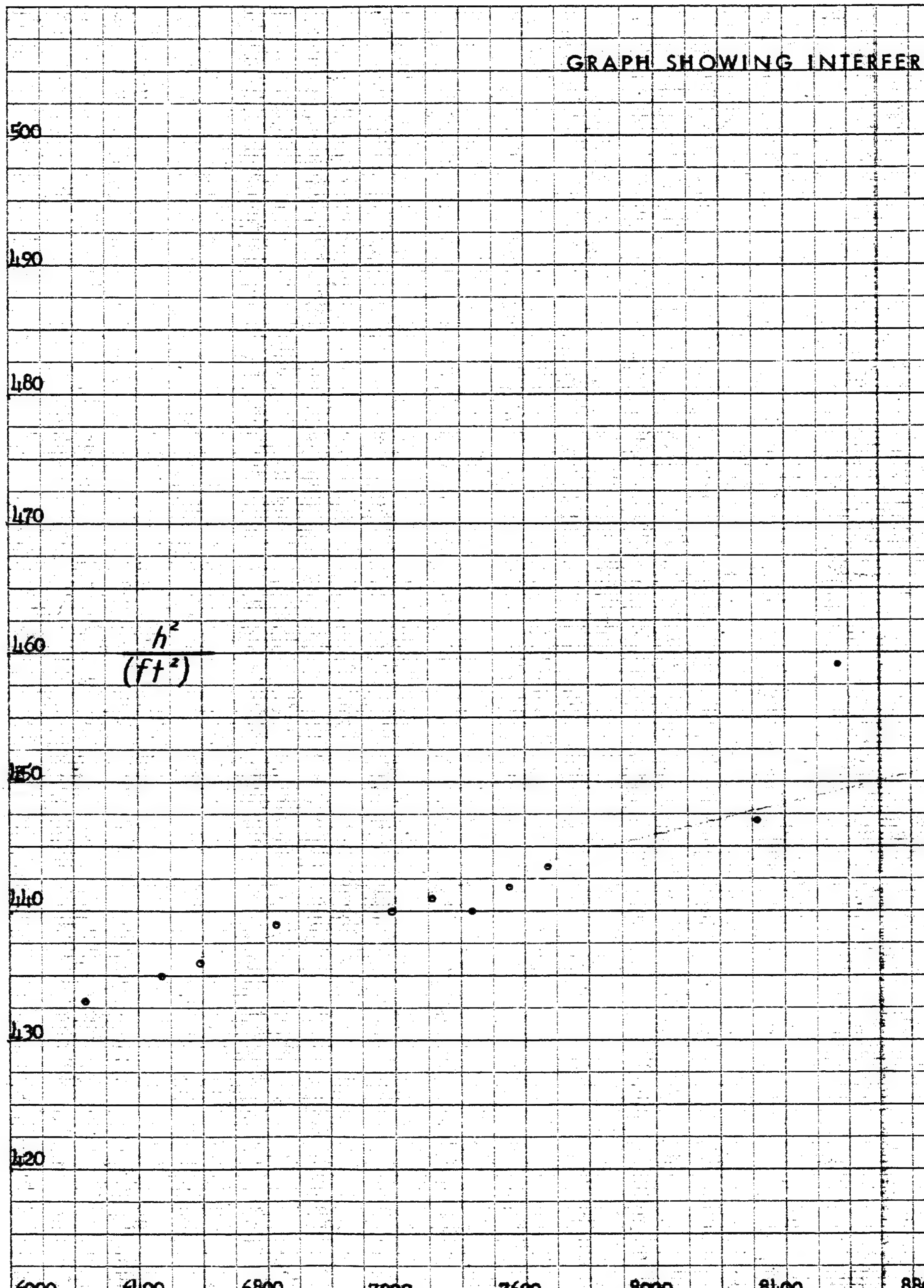
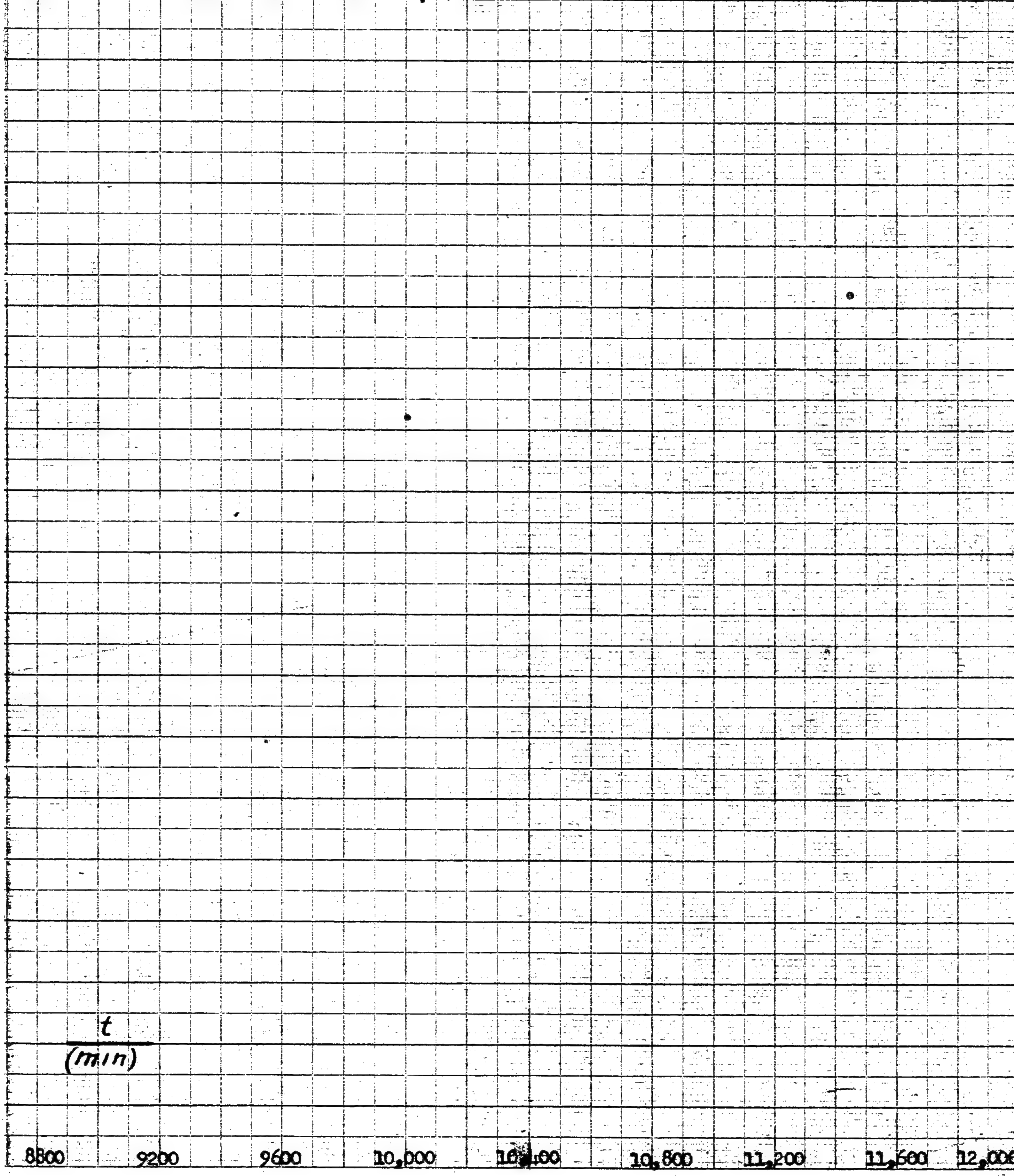

GRAPH SHOWING INTERFER
500
490
480
470
460
450
440
430
420
$\frac{h^2}{(ft^2)}$
6000
6400
6800
7200
7600
8000
8400

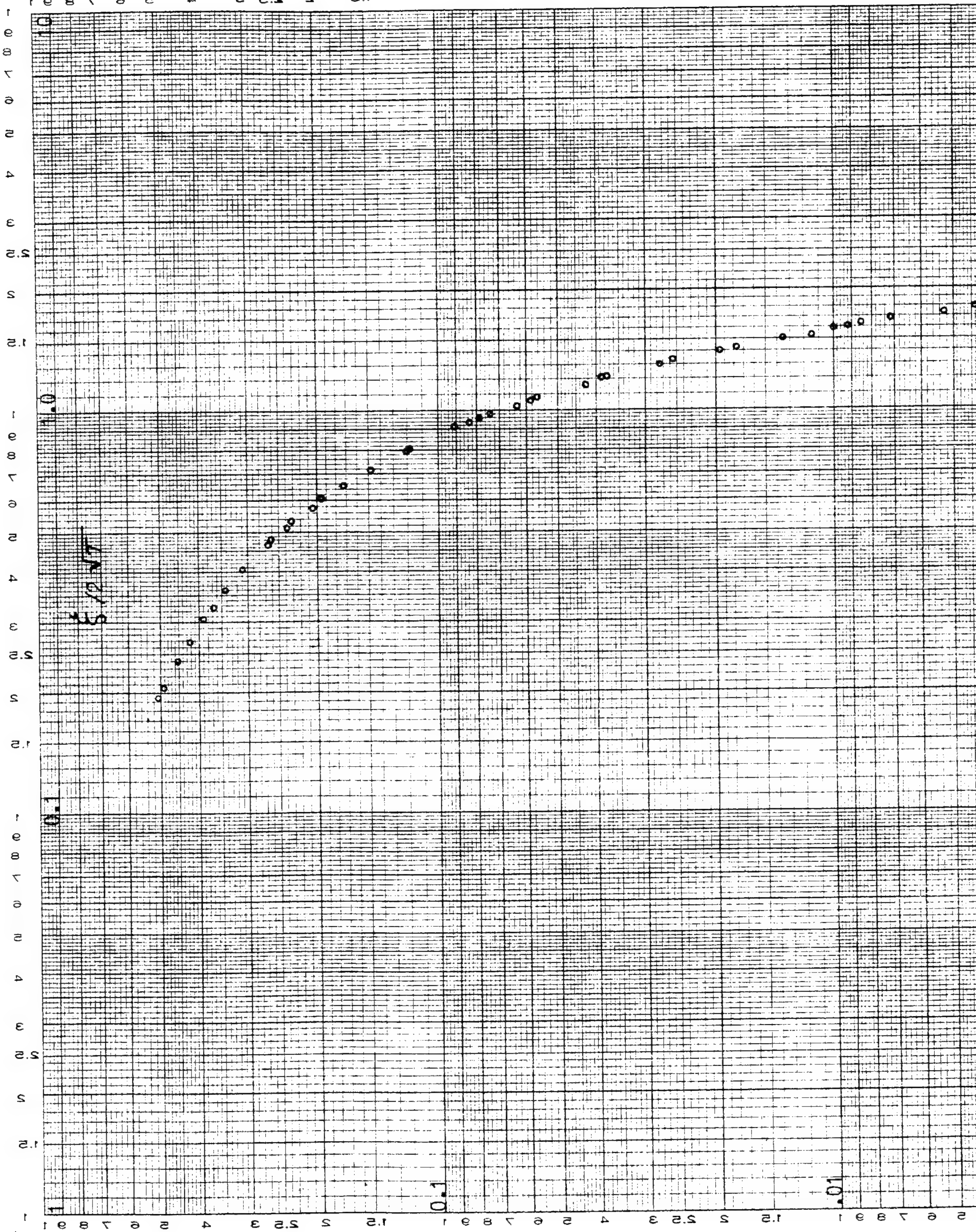
FIG. D-55b

FERENCE OF MARTY WELL ON P6, Contd.



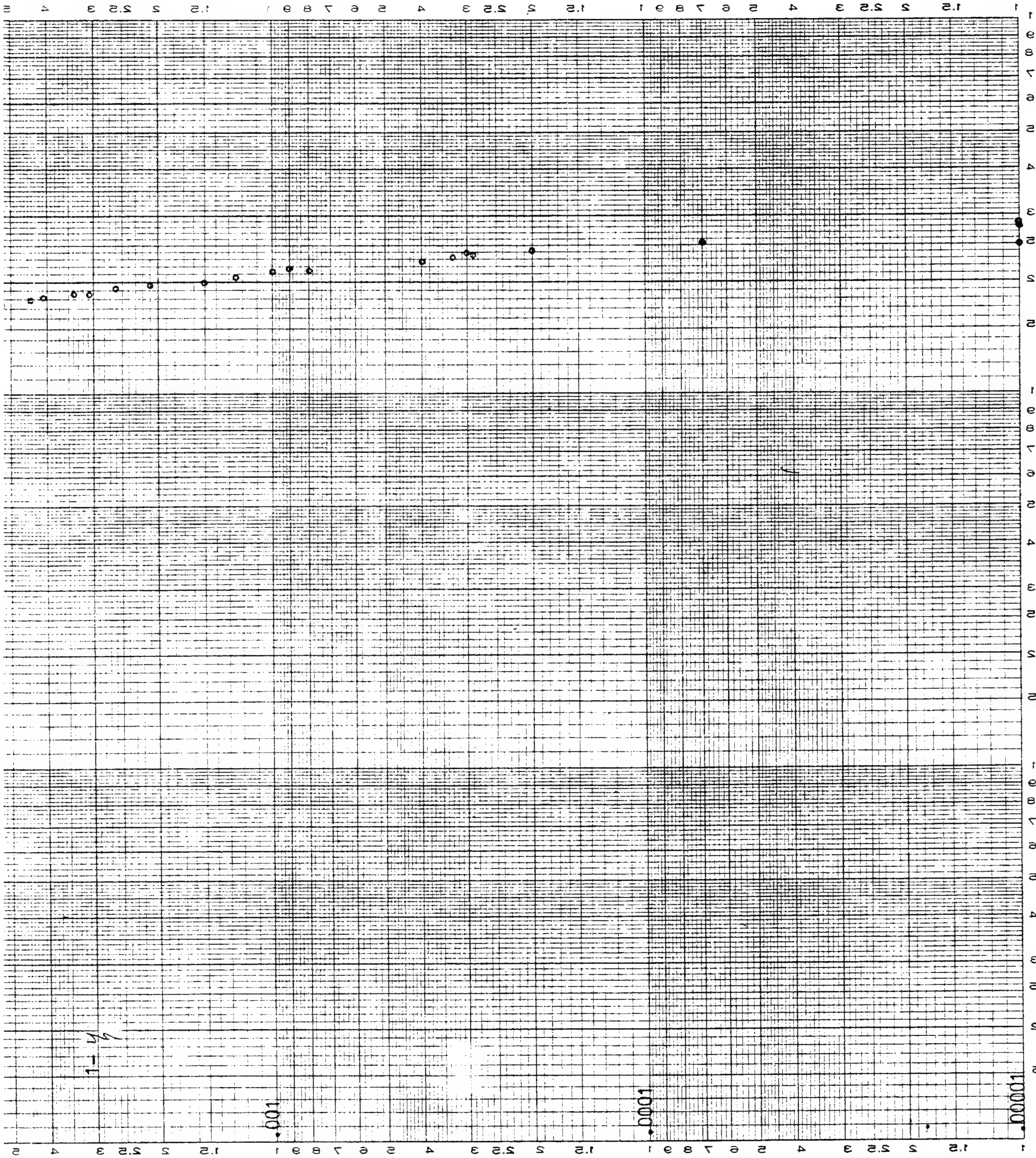
GRAPH OF WATER-TABL

FULL DRAWDOWN



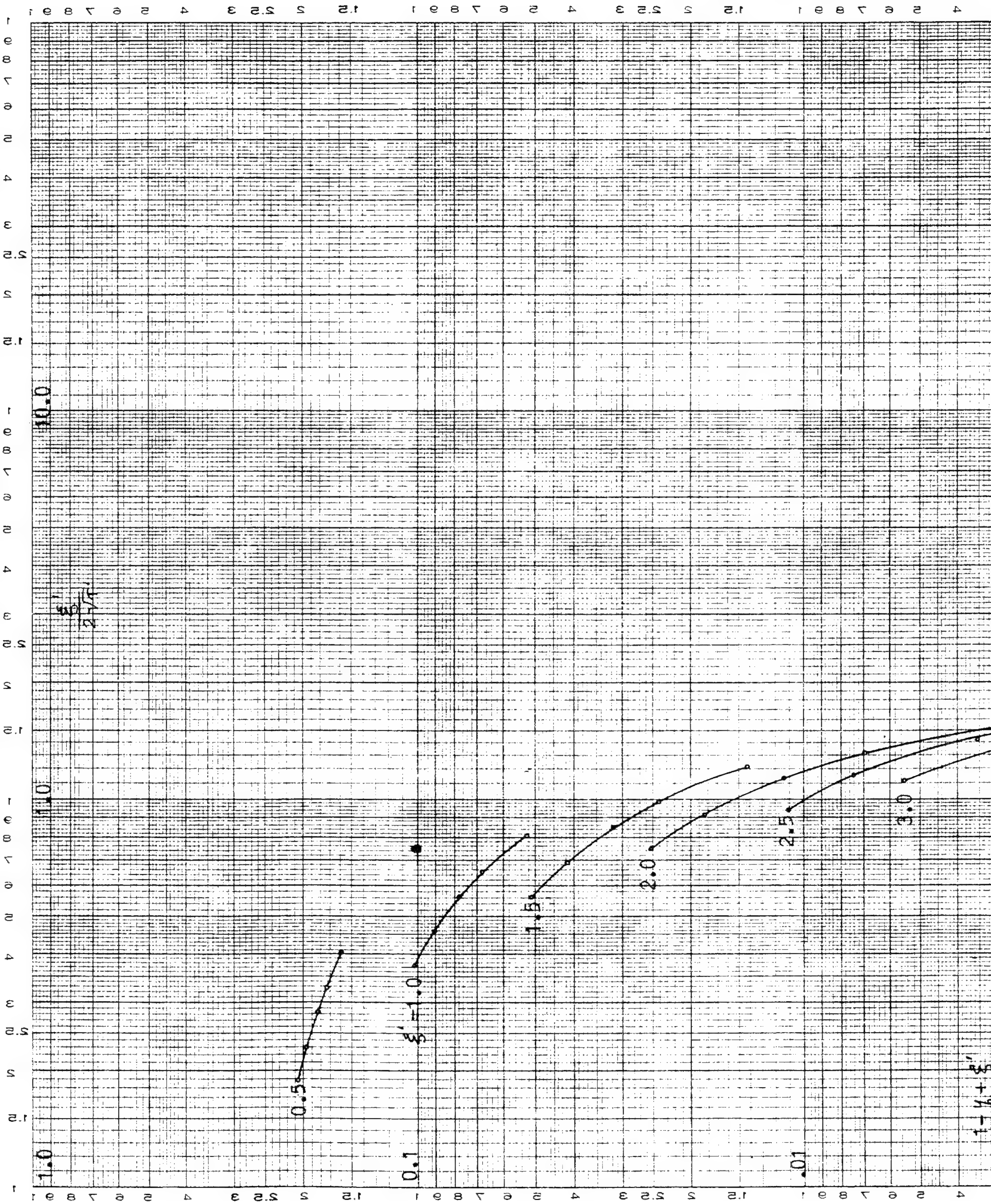
ABLE DECLINE NEAR TRENCH WITH
N IN HORIZONTAL AQUIFER

FIG. G-1



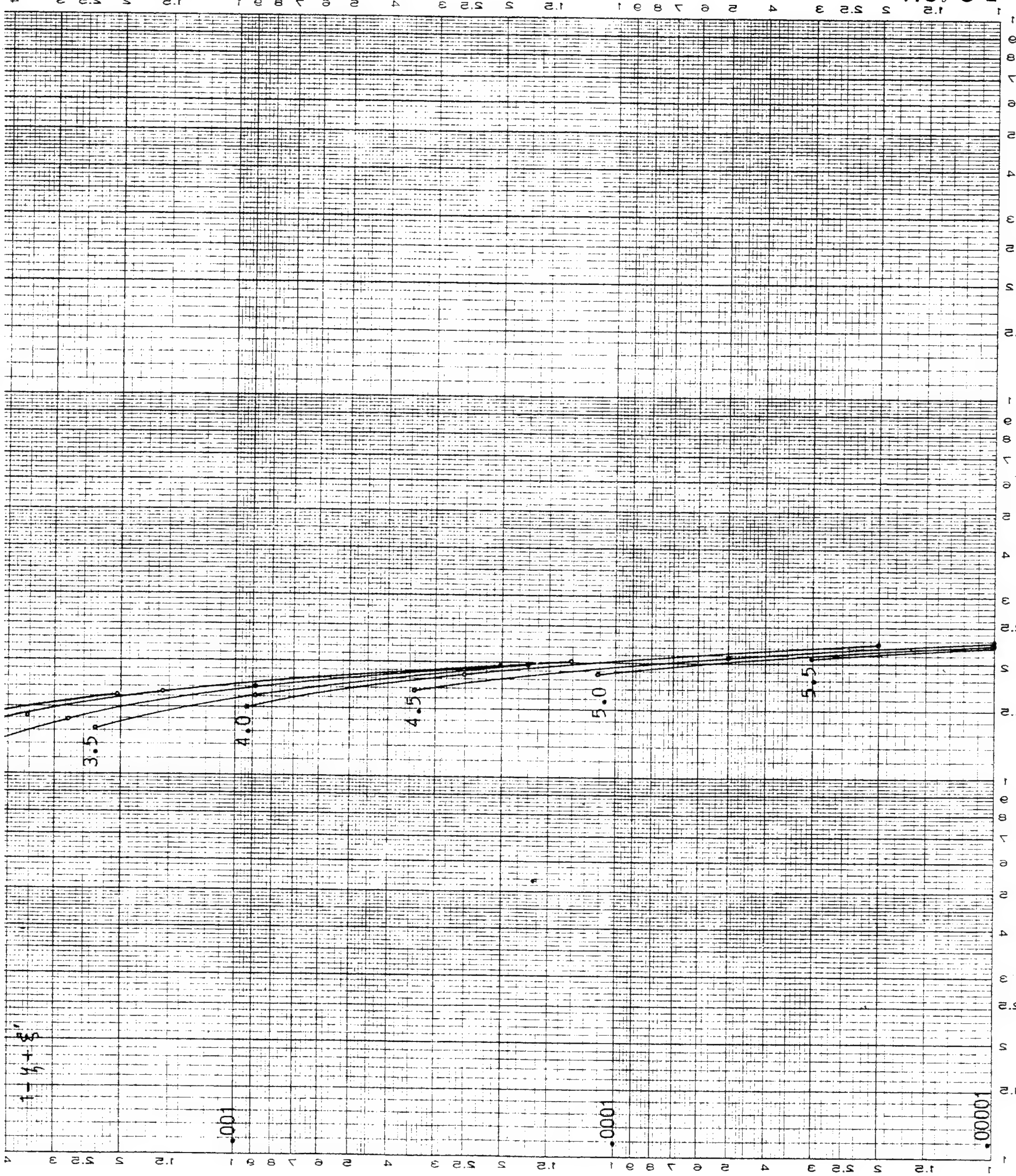
GRAPH OF WATER TABLE DECL

WITH FULL DRAWDOWN

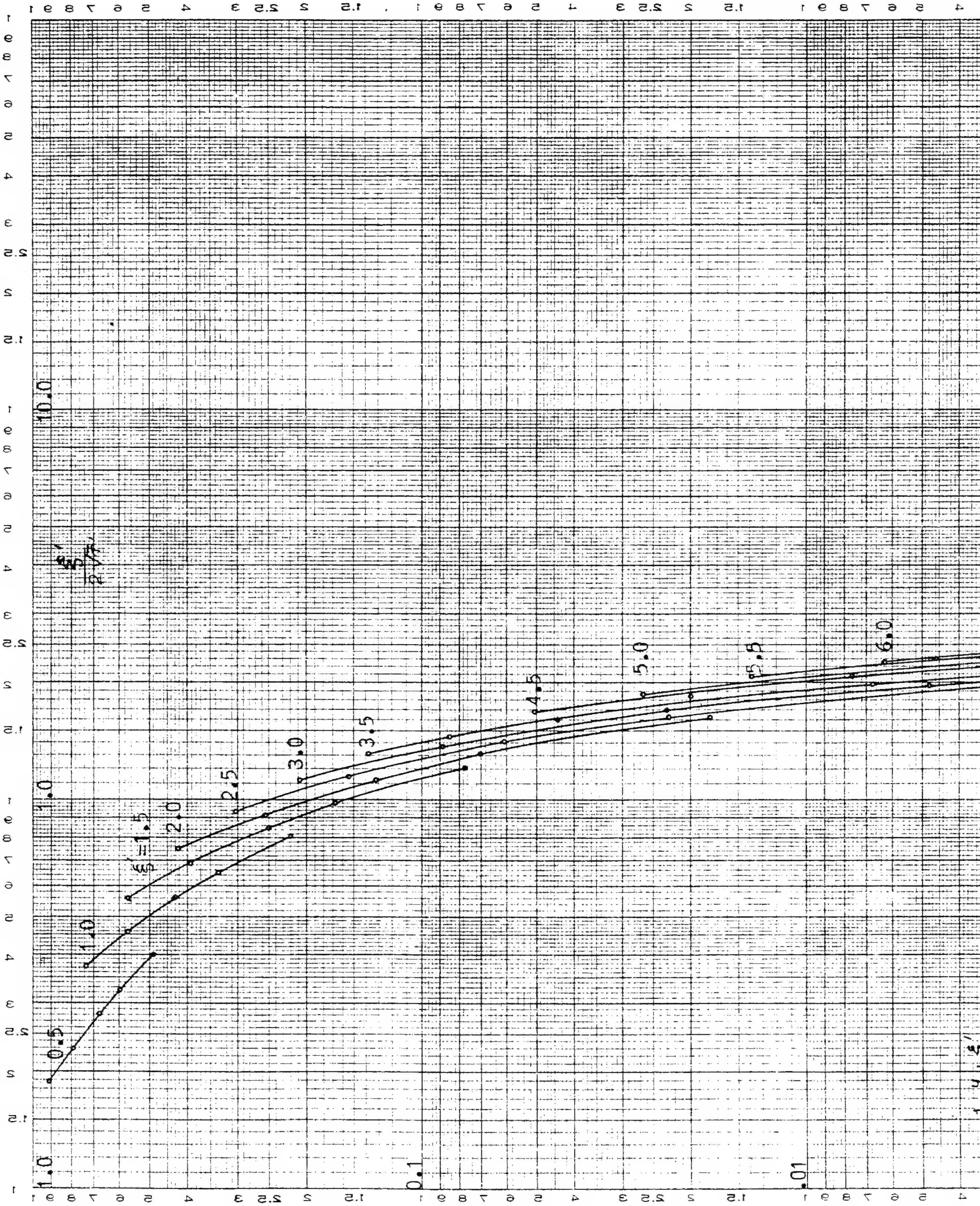


ECLINE UPSTREAM FROM TRENCH
/N IN INCLINED AQUIFER

FIG. G-2

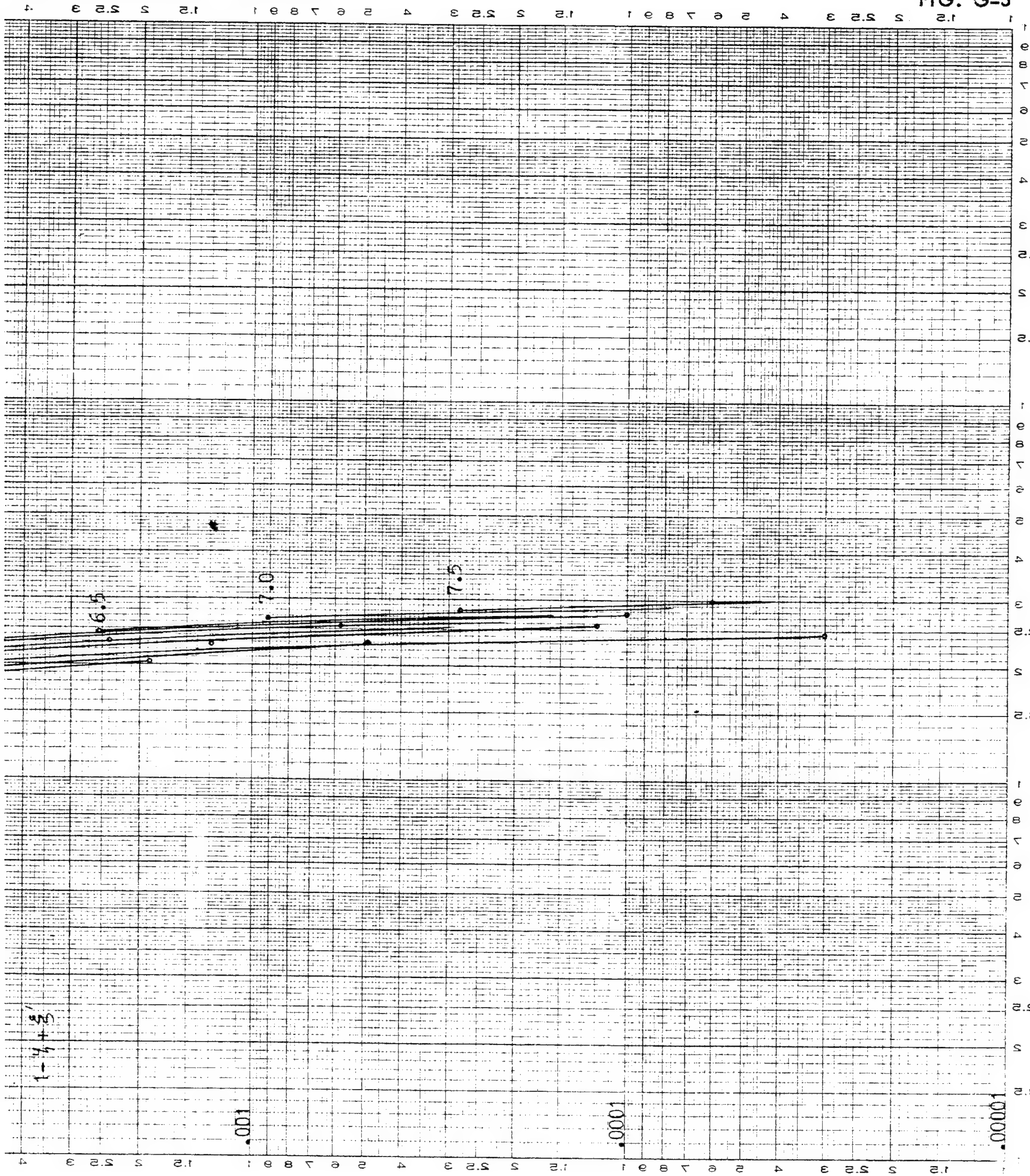


GRAPH OF WATER TABLE DECLI
WITH FULL DRAWDOWN



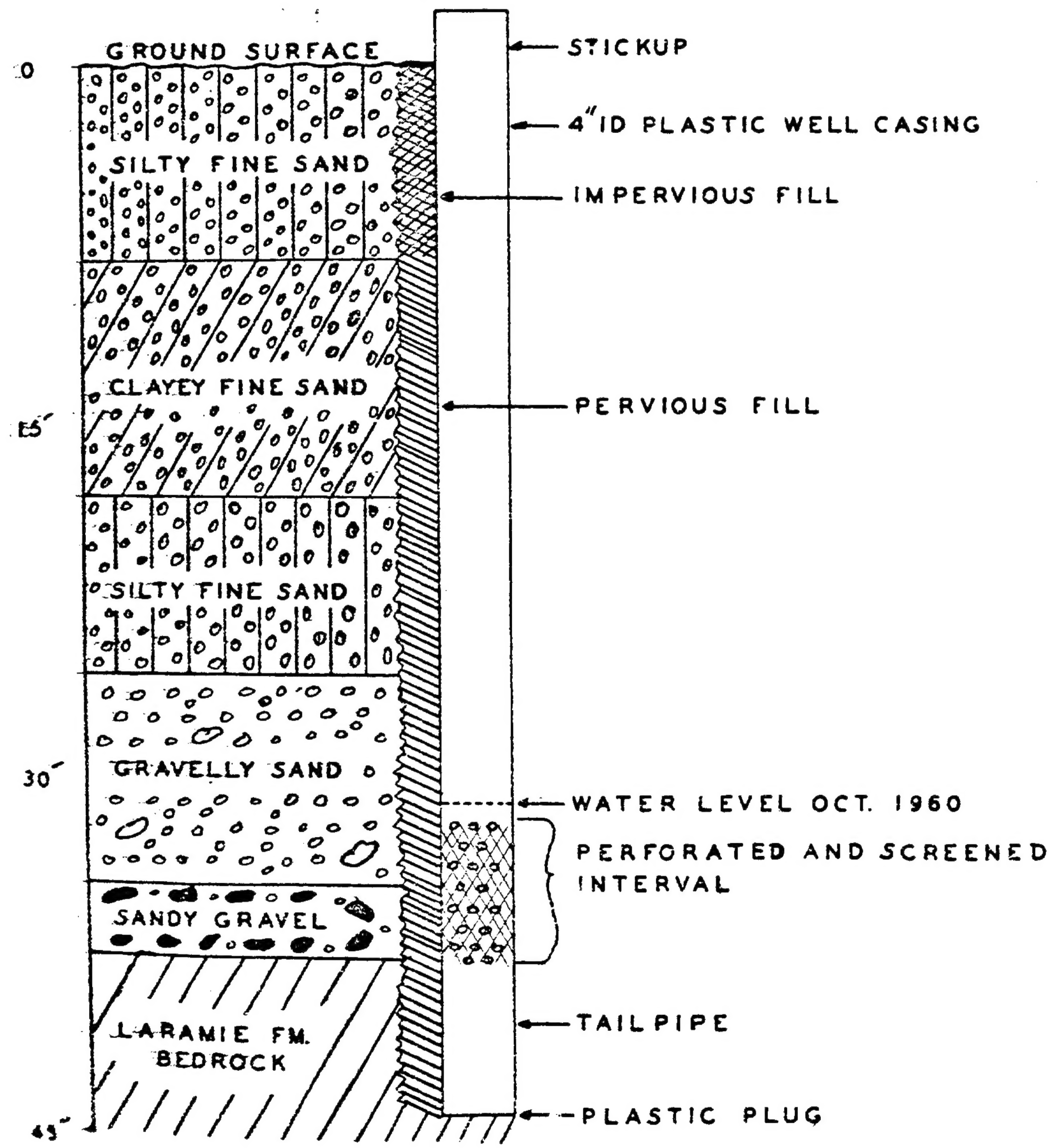
CLINE DOWNSTREAM FROM TRENCH
WN IN INCLINED AQUIFER

FIG. G-3



TYPICAL OBSERVATION WELL INSTALLATION

WELL 68



ROCKY MOUNTAIN ARSENAL

PROGRAM FOR RECLAMATION OF SURFACE AQUIFER

CORPS OF ENGINEERS

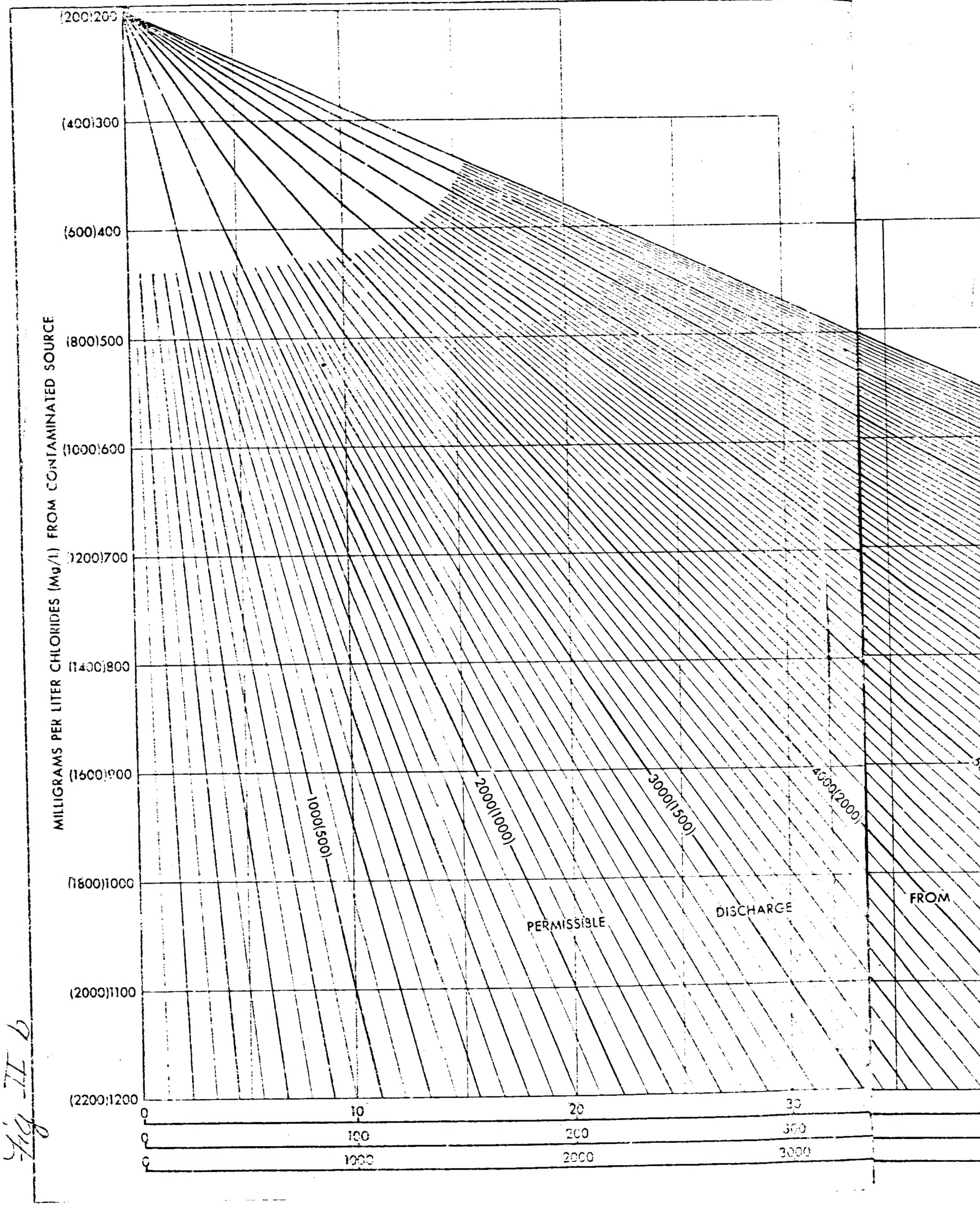


Fig II b

NOTES:

1. FOR RANGE 200-400 Mg/L CHLORIDES IN CONTAMINATED SOURCE USE CHART 1
2. FOR RANGE 400-2200 Mg/L CHLORIDES IN CONTAMINATED SOURCE USE CHART 2
3. FOR RANGE 2200-5200 Mg/L CHLORIDES IN CONTAMINATED SOURCE USE CHART 3
4. THE CHART IS BASED ON ZERO CONCENTRATION OF CHLORIDE IN THE RECEIVING STREAM. FOR ACTUAL VALUE OF PERMISSIBLE DISCHARGE FROM CONTAMINATED SOURCE IN GPM MUL CHART VALUE BY THE FACTOR FOR THE MONTH AND STREAM SHOWN IN THE TABLE ON THIS C THE FACTOR IS DETERMINED FROM THE AVERAGE CONCENTRATION OF CHLORIDE DURING EA
5. FOR EXAMPLE OF USE OF CHART, SEE CHART 1, Fig. IIa

U. S. ARMY

'CE USE CHART 1
IRCE USE CHART 2
JURCE USE CHART 3
HE RECEIVING STREAM.
TED SOURCE IN GPM MULTIPLY THE
N IN THE TABLE ON THIS CHART.
OF CHLORIDE DURING EACH MONTH.

FACTOR		
MONTH	SOUTH PLATTE RIVER	IRRIGATION DITCHES
JAN.	0.62	0.30
FEB.	0.64	0.17
MAR.	0.59	0.16
APR.	0.48	0.34
MAY	0.44	0.39
JUNE	0.53	0.43
JULY	0.60	0.48
AUG.	0.68	0.52
SEPT.	0.63	0.57
OCT.	0.54	0.39
NOV.	0.66	0.45
DEC.	0.60	0.35

ROCKY MOUNTAIN ARSENAL DENVER, COLORADO

PROGRAM FOR RECLAMATION OF SURFACE AQUIFER

PERMISSIBLE GPM DISCHARGE FROM CONTAMINATED SOURCE

CHART 2

JAN. 1961